FEASIBILITY STUDY AND COSTING OF PROPOSED POLLUTION CONTROL MEASURES IN THE HUMBER SEWERSHED

TECHNICAL REPORT #9

A REPORT

OF THE

TORONTO AREA WATERSHED
MANAGEMENT STRATEGY
STEERING COMMITTEE

JUNE 1986

TORONTO AREA WATERSHED MANAGEMENT STRATEGY STEERING COMMITTEE MEMBER AGENCIES

Ontario Ministry of the Environment

Metropolitan Toronto and Region Conservation Authority

Municipality of Metropolitan Toronto

Borough of East York

City of Etobicoke

City of North York

City of Scarborough

City of Toronto

City of York

Regional Municipality of Peel

Regional Municipality of York

Environment Canada

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Prepared by:
Paul Theil Associates Limited
Bramalea, Ontario

June, 1986

ACKNOWLEDGEMENTS

We would like to take this opportunity to express our sincere appreciation for the time expended by, and the comments received from the Project Committee. Furthermore, the efforts of Mr. W. Wong, Mr. R. Pickett, Mr. C. Ng and Mr. F. Engler in providing us with the necessary data and background knowledge to successfully complete this study are especially appreciated.

FEASIBILITY STUDY AND COSTING OF
PROPOSED POLLUTION CONTROL MEASURES
IN THE HUMBER SEWERSHED

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SUMMARY REPORT

INTRODUCTION

In an effort to maintain or upgrade water quality within the Toronto area watersheds, the Ministry of the Environment have commissioned a number of studies. This study, which was undertaken under the supervision of the Project Committee of the Toronto Area Watershed Management Strategy Study (TAWMS) deals specifically with the watersheds of the Humber River and the associated tributaries. Four specific tasks involving the feasibility and costing of various pollution control works were completed for this study. A brief description and the main objective(s) of each task is given below.

TASK 1 - Combined Sewer Overflow Control at Centralized Locations

Establish the location of and preliminary design of four storage facilities. Each of the four storage facilities is designed to reduce combined sewer overflows from the combined sewer system to the receiving body of water. The estimated costs for various sizes of each facility should also be determined. Three of the storage facilities are to be located within the City of York, at the downstream limit of the existing combined sewer systems(see Figure 1-1, locations 1, 2 and 3). The combined sewer overflows which are presently conveyed to the Black Creek will be stored in the facilities and ultimately discharged into the Black Creek sanitary trunk sewer. The fourth facility is to be located upstream of the Humber Water Pollution Control Plant (see Figure 1-1, locations 5, 6 and 7). The purpose of this facility is to attenuate sewage flows to the treatment plant, thereby reducing combined sewer overflows to the Humber River. The sizes of the above four facilities were determined in a previous study by the Ministry of the Environment. The feasibility of providing additional trunk sanitary sewer capacity adjacent to the Black Creek trunk sewer was also considered.

TASK 2 - Flow Control in Local Combined Sewers

Using underground detention tanks to store excess storm water, establish the estimated cost to reduce basement flooding within the City of York for selected design storm conditions. Stormwater runoff from grass and paved areas will be diverted to the underground detention tanks, stored in the tanks and released back into the existing sewer system at a rate which will not cause basement flooding. Only the areas presently served by combined sewers are to be considered. With the proposed underground detention tanks in place,determine the additional works required to reduce combined sewer overflows to the Black Creek. Construction of the above works will, as stated, reduce combined sewer overflows. In addition, the initial urban stormwater runoff will not enter the receiving body of water. This will further reduce the pollution loading to the Black Creek. The impact on the frequency of basement flooding and combined sewer overflows due to source reduction (disconnection of the roof downspouts from the existing combined sewer) was also considered.

TASK 3 - Separation of Combined Sewers

Estimate the cost of sewer separation within the combined sewer catchments in the City of York. Separation of the combined sewers will reduce existing basement flooding problems. The estimated cost and the feasibility of disconnecting roof downspouts and foundation drains from the existing combined sewers was also determined.

TASK 4 - Stormwater Detention Ponds

Examine the engineering feasibility and provide cost estimates for constructing retention ponds at or near the point of storm sewer outlets to the Humber River or its tributaries from four catchment areas. The catchment areas which were considered are primarily industrial. The effect of the proposed ponds for controlling the pollution of

stormwater runoff to the Humber River and its tributaries was determined in a separate TAWMS project. Three of the proposed retention ponds are to be located in Etobicoke while one pond is to be located in North York.

SUMMARY OF FINDINGS

The first three tasks are somewhat interrelated. Installing underground tanks to store the excess stormwater which cannot be conveyed by the existing combined sewer (Task 2) reduces basement flooding as does sewer separation (Task 3). The storage facilities as proposed in Task 1 are similar to the works for reducing combined sewer overflows in Task 2. Furthermore, the proposed Black Creek sanitary trunk sewer and the storage facility upstream of the Humber Water Pollution Control Plant (Task 1) are only required if the existing flow regulators within the City of York combined sewer system are modified to divert more flow to the Metropolitan trunk sewer system. Under this condition, the proposed storage tanks in Task 1 would be reduced in size. The reader is therefore reminded that the cost estimates as presented below provide a range of costs for various alternatives that may, in some cases, be implemented to achieve similar goals.

Following the Summary Report is a brief description of the main advantages and disadvantages of the Inlet Control Method (Task 2) and sewer separation (Task 3).

A brief summary of the estimated costs for the proposed works follows.Descriptions of the sizes of the proposed works and the benefit of constructing the proposed facility is also given.

The cost estimates were based on 1984 construction costs and include installation, restoration, operation, maintenance and replacement costs, land costs and a 20 percent engineering and contingency allowance. All annual costs were converted to capital costs, using a 70

year time period and a net interest rate (interest rate minus inflation rate) of 7 percent. Where applicable, both capital costs and capital costs plus converted annual costs (shown in brackets) are given. Commensurate with the nature of the study, all costs are preliminary and are therefore subject to revision at the detail design stage.

TASK 1

Cost estimates were prepared for the construction of either above ground (open basin) or below ground (underground tanks) facilities to reduce combined sewer overflows to the Black Creek. The locations for the proposed facilities are at Hyde Avenue and at Rockcliffe Boulevard (see Figure 1-1, locations 1, 2 and 3). A range of cost estimates were prepared for various sizes and types of facilities.

An estimate has also been made of the expected average annual cost of cleaning of tanks and basins, as well as maintenance of monitoring equipment and valves.

Construction of two facilities, one at Hyde Avenue and one in the area of Rockcliffe Boulevard (i.e. the storage tanks at locations 2 and 3 would be combined) with a combined volume of 8,000 m^3 would reduce combined sewer overflows to the Black Creek to approximately 15 events* per year. The cost to construct open basins of this volume would be approximately $690,000 ($870,000). Underground tanks could be constructed for $1,325,000 ($1,500,000). Enlarging these two facilites to provide a combined storage volume of 52,000 m^3 would reduce the combined sewer overflows to one event per year. The estimated costs increase to $1,875,000 ($2,106,000) for open basins and $4,688,000 ($5,063,000) for underground tanks. The estimated cost to treat the combined sewer overflows for a 70 year period is $207,000.

Cost estimates for the proposed 4,200 m^3 storage facility upstream of the Humber Water Pollution Control Plant indicated that an open basin

* Ministry of the Environment, "Synopsis of TAWMS Task PCC2, Combined Sewer Analysis in Humber Study Area", July 1984

could be constructed within the treatment plant site for $310,000 ($393,000). A facility located upstream of the plant (see locations 5 and 6,Figure 1-1) could be constructed for about $550,000 ($640,000) to $621,000 ($720,000), depending upon the selected location. The proposed sanitary trunk sewer which would parallel the existing Black Creek sanitary trunk sewer could be constructed for $1,450,000.

TASK 2

Cost estimates for reducing basement flooding for areas within the City of York which are still serviced by combined sewers were prepared. Estimates were prepared for the remedial works which would be required to eliminate basement flooding for each of the 2, 5 and 10 year design storms.

As stated in the Terms of Reference, the principal component of the remedial works is to be underground detention tank storage. The results showed that a 2 year level of protection against basement flooding within the City of York could be provided for $1,600,000 ($1,810,000). A 5 year level of protection would cost approximately $9,600,000 ($10,020,000), and a 10 year level $16,600,000 ($17,020,000). The above cost estimates assume that no water will be stored on the roadways for the given design storm event. Previous experience from similar types of projects by our firm has indicated that roughly one third of the excess storm water can typically be stored on the roadways with minimal inconvenience to the adjacent homeowners. Storing water on the roadways would have the effect of reducing the detention tank sizes and associated costs.

The most cost effective method for reducing combined sewer overflows to the Black Creek is to provide storage facilities at the downstream limit of the catchments. The location of the facilities would be identical to those described in Task 1. Various options for providing different levels of basement flooding protection and reduction of combined sewer overflows to the Black Creek were considered. As an indication of the total cost, the estimate to provide a 10 year level

of protection against basement flooding and reduce combined sewer overflow events to the Black Creek to an average of one per year is approximately $18,840,000 ($19,350,000).

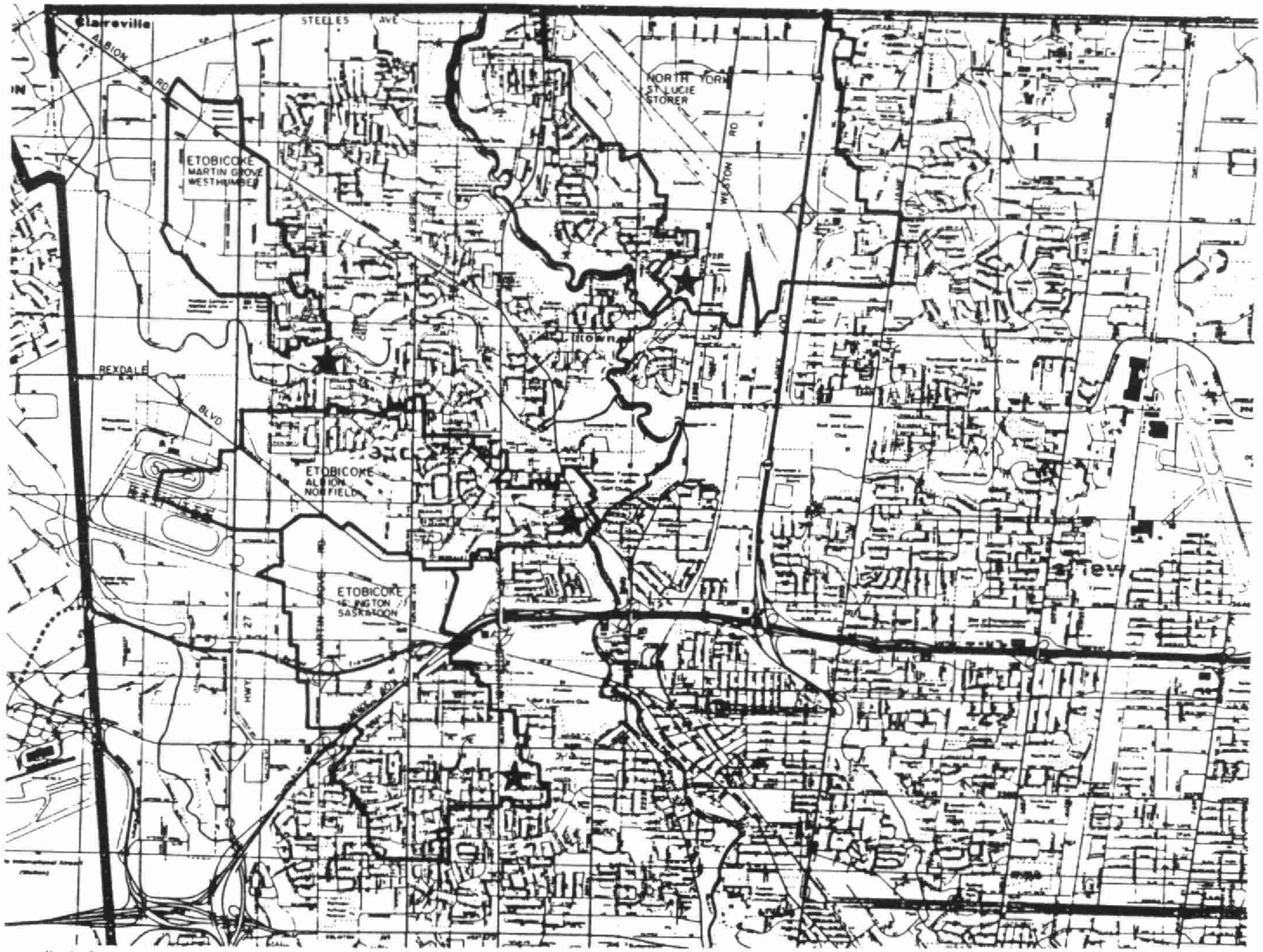
TASK 3

In this Task the cost to separate the remaining areas within the City of York which are still serviced by combined sewers was estimated. The total cost was found to be $86,500,000. The feasibility and estimated costs to disconnect the foundation drains and roof down-spouts from the existing combined sewer were also determined. It was found that due to lot grading constraints, only the front yard down-spouts could be disconnected. The estimated cost is $2,500,000. The estimated cost to disconnect the foundation drains from the existing combined sewer and reconnect the drains to the proposed storm sewer is $127,000,000.

TASK 4

The four locations at which the feasibility of constructing retention ponds has been studied are shown on the location plan (Plate 4-1). Cost estimates were prepared for three different sizes of the facilities at each location. The catchment, associated required detention volume and estimated cost are given below for the largest storage requirements (20 mm of runoff storage per hectare of catchment area).

			Capital Cost	Capital Cost + Maintenance
1.	St. Lucie Drive and Storer Drive;			
		154,000 m^3;	$1,182,000.	($1,370,000)
2.	Martingrove and West Humber;			
		52,800 m^3;	$ 842,000.	($ 914,000)
3.	Albion and Norfield;			
		96,000 m^3;	$ 795,000.	($ 934,000)
4.	Islington and Saskatoon;			
		130,000 m^3;	$ 425,000.	($ 590,000)



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

STORMWATER RETENTION PONDS CATCHMENT AREAS

legend

★

The estimated costs range from a low of $654 ($908) per hectare of drainage area for the Islington and Saskatoon catchment to a high of $3,186 ($3,461) per hectare of drainage area for the Martingrove and West Humber catchment.

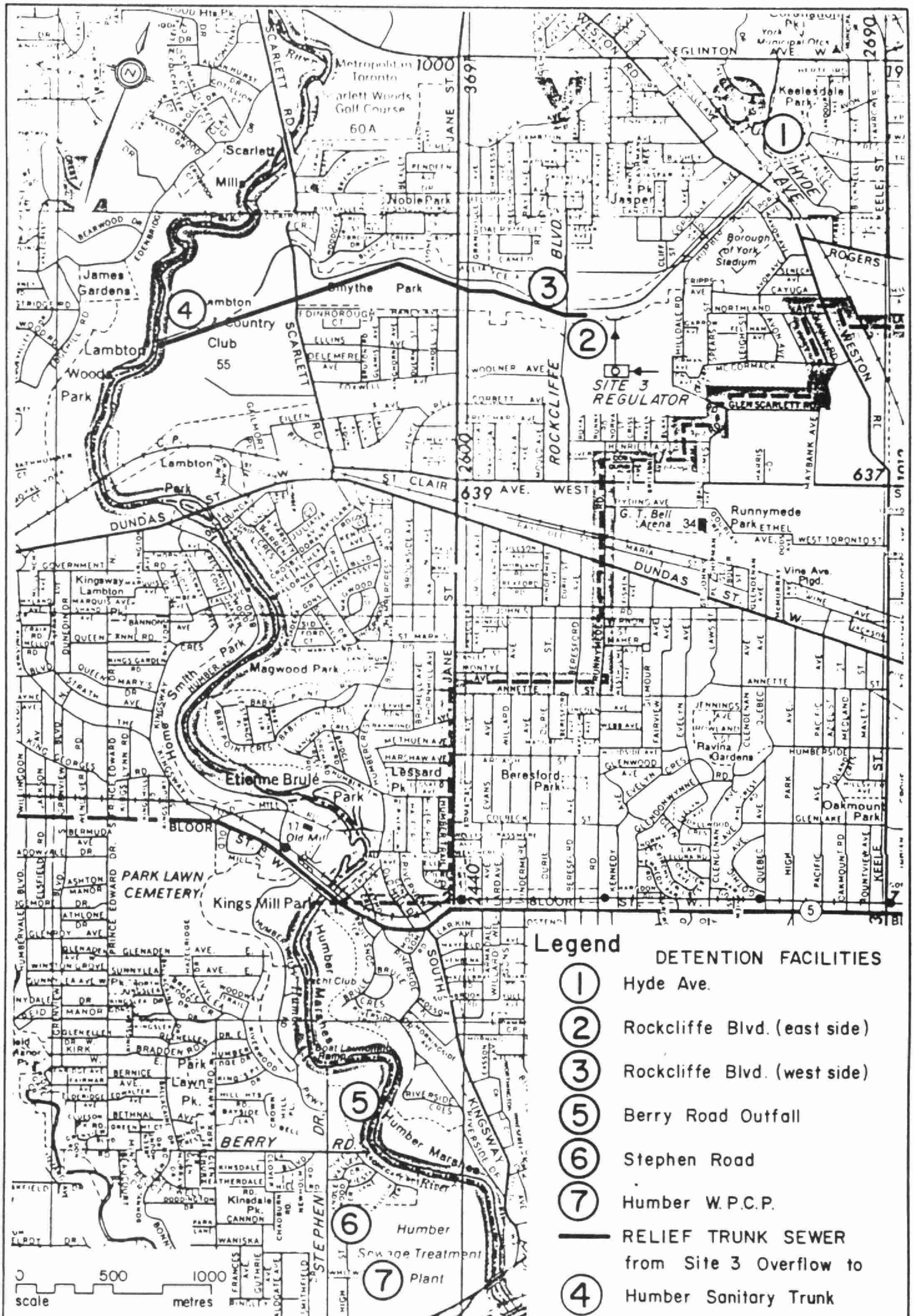
CONCLUSIONS

The feasibility of constructing remedial works to reduce combined sewer overflows, store stormwater runoff and eliminate basement flooding was established for numerous alternatives. Preliminary cost estimates were prepared for each feasible alternative. The following conclusions may be drawn from the study:

i) It is technically feasible to construct either open basins or underground tanks at Hyde Avenue and the Rockcliffe Boulevard area which are of sufficient size to reduce combined sewer overflows from the City of York sewer system to about one event per year. The final selection of the location, size and type of facility (open basin or underground tank) will depend upon several non-monetary factors which include the desired degree of combined sewer overflow reduction, aesthetics, safety and the existing land uses of the proposed sites.

ii) The proposed storage tank upstream of the Humber Water Pollution Control plant could be constructed to its ultimate size in one of three locations, as shown in Figure 1-1. According to M.O.E. this will effectively eliminate combined sewer overflows to the Humber River from the Berry Road regulator.

iii) The reduction of basement flooding for the areas within the City of York which are served by combined sewers can be achieved by constructing underground detention tanks or separating the existing combined sewers. A comparison of the relative costs indicates, however, that it is significantly less expensive to reduce basement flooding by installing underground detention tanks as opposed to separating the sewer system.



LOCATION PLAN

figure 1-1

iv) A combination of underground detention tank storage where flooding problems exist and storage facilities at the overflow locations within the combined sewer system will achieve both goals of reducing basement flooding and reducing combined sewer overflows to the Black Creek. This alternative is considerably less costly than a sewer separation program which may or may not achieve the latter objective of reducing combined sewer overflows to a specified level. Furthermore, by eliminating the need for installing separate storm sewers, the significant pollution of receiving waters that occurs from direct discharge of urban runoff is avoided.

v) The feasibility of constructing retention ponds to store 10, 15 or 20 mm average runoff over the catchment area for four predominantly industrial areas within the cities of Etobicoke and North York was determined. Preliminary designs and cost estimates for each of the four ponds were completed.

As stated in the Terms of Reference, recommendations for this study are not to be given by the consultant. Recommendations will be forthcoming from the management of the TAWMS after this study has been incorporated into the watershed management strategy plan.

Respectfully submitted

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SEWER SEPARATION VERSUS INLET CONTROL

Combined sewers have two inherent problems; 1) basement flooding when runoff exceeds pipe capacity, and 2) combined sewer overflow (C.S.O.) when flow exceeds capacity of regulators, resulting in water pollution.

The traditional solution to these problems has been to carry out a sewer separation programme, such as presented in Task 3 of this study. Another solution that has gained considerable interest in recent years is the Inlet Control Method (I.C.M.). Implementation of either of these methods will generally overcome the problems, although not to the same degree. The following is an outline of the two methods as covered in Tasks 2 and 3 of this study, and a description of their operation and performance.

The existing combined sewer network within the City of York carries sanitary waste water and flow from runoff events. A series of flow regulators controls the flow that goes to the Humber W.P.C.P., diverting excess flow during major runoff events to Black Creek. Since approximately 60 percent of runoff events do not exceed the capacity of the regulators, and the initial runoff of the remaining events is also conveyed to the W.P.C.P., the most significant part of the pollutants from surface runoff is being treated at the plant.

Pollutants from surface runoff have been identified, through extensive monitoring by U.S.-.E.P.A. (Reference 1), as consisting mainly of heavy metals (copper, lead and zinc). Some of the metals are present often enough and in high enough concentrations to be potential threats to beneficial use. In many instances concentrations at storm sewer outlets were found by E.P.A. to exceed their criteria for receiving water quality criteria. The monitoring carried out by TAWMS support these findings. With sewer separation pollutants from surface runoff will be discharged directly to Black Creek and the Humber River, whereas now pollutants, for the most part, go to the W.P.C.P.

Sewer separation as has been proposed for the City of York consists of new storm sewers within roadways with capacity to handle runoff from paved surfaces for the 2 year storm, similar to the criteria being used by the City of Toronto. Roof water will continue to be discharged to the combined

sewers, and as such only partial separation can be achieved and C.S.O. will continue to occur, although less frequently. With the sewers being designed for a 2 year storm only, basement flooding during higher intensity storms is still possible. The estimated cost for sewer separation is $86.5 million.

With sewer, separation, work must commence from an open watercourse such as the Black Creek and proceed upstream to the areas affected. Frequently it is the areas with the smallest pipes in the upper reaches that suffer from flooding most often. Before relief can be provided to these areas, disruption to residents and traffic often occur on streets which have not experienced flooding problems. With the inherent high cost of sewer separation, budget constraints usually result in considerable time lag before relief is available to the areas most affected.

Although sewer separation does provide significant protection against base- ment flooding, the limited capacity of the storm sewers will result in surface ponding in sags in roadways when runoff exceeds the pipe capacity. Since in many cases the ground level around houses is not much above curb elevation, flooding from overland flow is quite possible.

There is another solution that has gained considerable interest in recent years - the inlet control method. This method which has been analyzed as part of Task 2 of this report was first introduced in the City of York in 1976, and has since been applied in several other municipalities. It is based on the principle that combined sewers work well provided surcharging does not occur. Flooding problems only show up when runoff exceeds pipe capacity. By limiting the inlet capacity to the pipe capacity, surcharging can be avoided. The problem is then reduced to providing temporary storage for excess surface runoff, when such a condition occurs. In Task 2, storage is provided by underground detention. The cost of providing underground detention for surface runoff for 2, 5 and 10 year storms is presented. Past projects in the City of York of this type have been designed on the basis that runoff from storms up to 10 year frequency are accommodated in underground tanks, reducing considerably the risk of flooding due to overland flow. Since the inlet capacity to the

existing combined sewer will be limited to prevent surcharging, sewer back-up for any storms is unlikely to occur. In this manner, the inlet control method will provide a higher level of flooding protection than available with conventional sewer separation. In addition, it can provide immediate relief to priority areas, since it is not dependent upon new storm trunk sewers.

The detention facilities provided with the Inlet Control Method will reduce the frequency of combined sewer overflows to some degree, depending upon the criteria selected for the design. By providing further detention at the location of the current overflow regulators, it is possible to reduce the frequency of C.S.O further to almost any degree. The total estimated cost for a 10 year system within roadways plus detention at regulators, sized to reduce C.S.O. events to an average of one per year, is $18.8 million, or $19.4 million if allowance is made for maintenance for a period of 70 years. Flows that have been detained will slowly be released over a period up to 24 hours thus balancing the flows to Humber W.P.C.P. and preventing serious overloading. The volume of flow diverted to the treatment plant,rather than overflowing to Black Creek as at present, represents an annual increase of 0.1 percent and as such, any increase in the cost of treatment will be insignificant.

In summary, for about 22 percent of the cost of sewer separation, the inlet control method in comparison with sewer separation will:

- provide a higher level of protection against basement flooding
- provide immediate relief to priority areas without need for expensive storm trunk facilities
- with downspouts left connected, provide a greater reduction of combined sewer overflow. This will occur as a result of attenuation of the peak flows through the detention tanks.
- capture pollutants in storm runoff, for conveyance to Humber W.P.C.P. rather than discharging the storm runoff untreated to Black Creek and Humber River.

Current M.T.C. policy provides subsidies for construction of storm sewers "necessary for the replacement of, or to increase the capacity of an existing storm sewer for road drainage purposes". The subsidy is based on

the diameter of the sewer, with full subsidy for pipes less than 675 mm in diameter, decreasing as the sewer size increases. This policy pre-dates the application of the Inlet Control Method.

Some municipalities using I.C.M. have successfully argued that where the outlet pipe from the detention tank is less than 675 mm in diameter, full subsidy for the works should be available. Others have only received subsidy in strict conformity with the basic formula (i.e. diameter of detention tank), resulting in only minor subsidy for the facilities. As the underground tank is used in the same capacity as a storm sewer (i.e. both methods reduce storm drainage problems along local streets), the detention tanks should be subsidized to the same extent as storm sewers.

This inequity has led some municipalities to prefer sewer separation, favouring the higher grant, although net costs to the municipality may still be higher. This action will result in higher costs being borne by M.T.C. and ultimately the taxpayer.

Since overall emphasis should be made to promote any alternative that will provide better service at lower cost, it would be desirable if such alternatives received a higher rate of subsidy to encourage cost savings. An example of this is the policy of U.S.-E.P.A. whereby innovative solutions approved by E.P.A. as being technically superior and/or less costly will receive a higher rate of subsidy. Furthermore, no subsidy is provided for conventional solutions unless an engineering report has verified that no innovative solution is practical. This policy has had the effect of many improved techniques being developed and successfully applied, rewarding progress rather than penalizing it.

References:

1. United States Environmental Protection Agency (Dec. 1983) "Results of the Nationwide Urban Runoff Program".

2. Paul Theil Associates Limited (unpublished paper): "Sewer Separation - Why does it not meet the expectations?"

SECTION 1

TASK 1

Combined Sewr Overflow Control at Centralized Locations

TASK 1

COMBINED SEWER OVERFLOW CONTROL AT CENTRALIZED LOCATIONS

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TASK 1

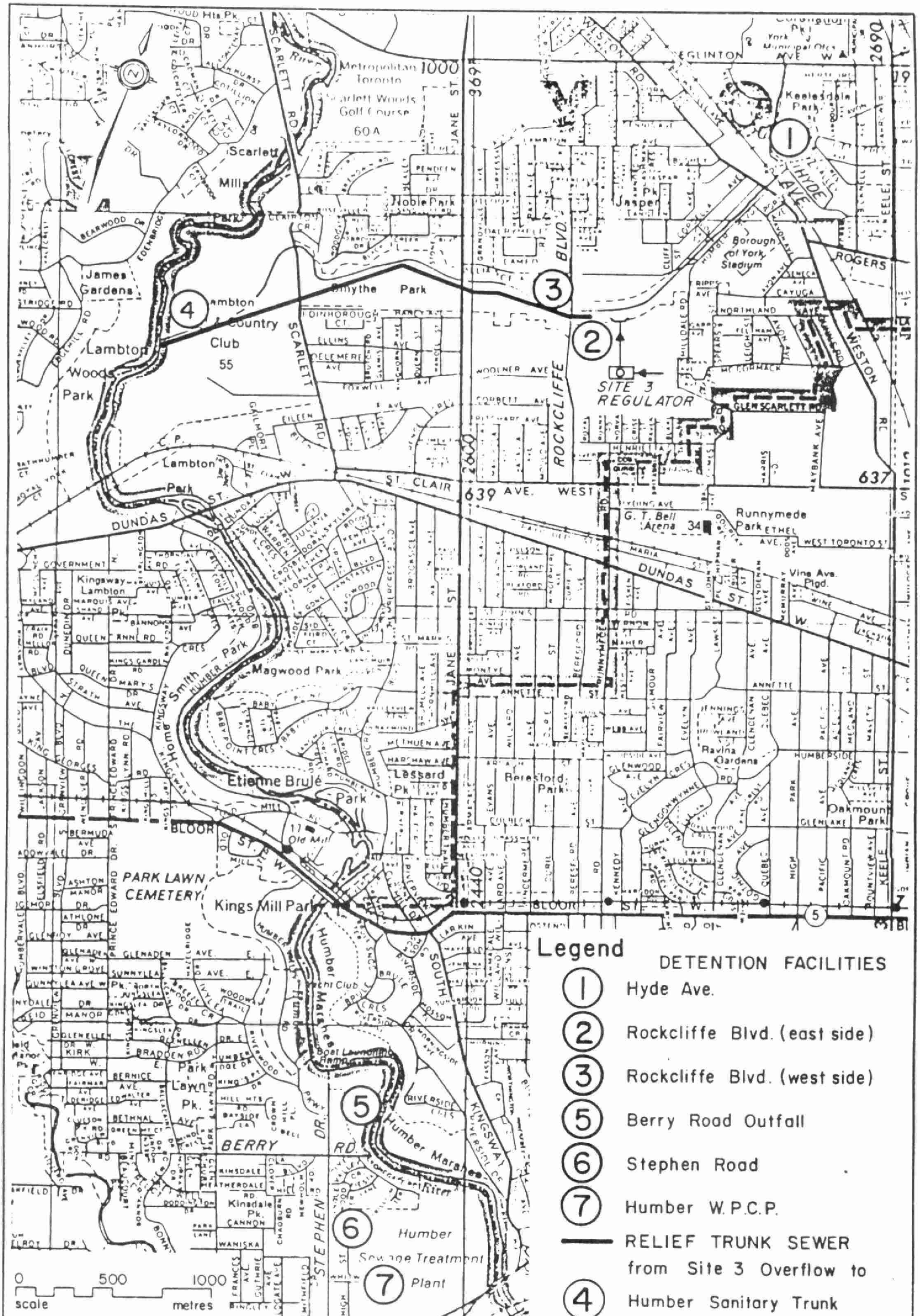
COMBINED SEWER OVERFLOW CONTROL AT CENTRALIZED LOCATIONS

1 - INTRODUCTION

Task 1 is related to another TAWMS study identified as: "TAWMS TASK PCC2 - Combined Sewer Analysis in Humber Study Area", in which the control of combined sewer overflow (C.S.O.) to Black Creek and Humber River from existing regulators has been studied by computer simulation. The flows and detention volumes required for various degrees of C.S.O. control have been determined in this study.

Task 1 of this study examines the engineering feasibility and costing of these C.S.O. flow control measures. The proposed works include four detention facilities for C.S.O, three of which are to be located adjacent to the Black Creek and one in the vicinity of the Humber Water Pollution Control Plant (HWPCP) and a sanitary trunk sewer located between the Humber River and an existing flow regulator in the City of York (see Figure 1-1).

The various alternatives are shown schematically on the plates. The cost estimates as presented in Section 5 are based on late 1984 construction costs and include all the required inlet, outlet and overflow piping. Allowances for engineering and contingencies have been included.



LOCATION PLAN

figure 1-1

For open basins, the cost includes for excavation, disposal of surplus material, sodding, gravel driveway for vehicular access, fencing and perforated underdrains to prevent standing water between intermittent uses.

The cost for underground tanks includes excavation, construction of reinforced concrete structure, backfill, disposal of surplus material, and restoration of areas disturbed under construction. It also includes the cost of an entrance building, mechanical ventilation and gravel driveway for vehicular access. No allowance has been made for land costs, since all the proposed facilities are within easements or lands owned by the public.

Based on the soils information available, we have assumed that all excavation can be carried out without special dewatering and tight sheeting, except for the proposed sewer from Rockcliffe Boulevard to the Humber sanitary trunk sewer, where unstable ground conditions are anticipated. We have also made allowance for utility relocation in the few cases this will be required.

The drains from tank or basin flow to the Metropolitan Toronto trunk sanitary sewer are sized to enable emptying within 24 hours.

2 - DETENTION FACILITIES

In accordance with the Terms of Reference, the feasibility of constructing C.S.O. detention facilities has been investigated for the following general locations, as shown on Figure 1-1.

- Hyde Avenue
- Rockcliffe Boulevard
- Berry Road
- Humber Water Pollution Control Plant.

Although underground tanks no doubt are preferable from the adjacent property owners' point of view, open basins are technically feasible. As such we have prepared cost estimates for both types of detention facility, except for the facility east of Stephen Drive, north of the Humber W.P.C.P. where an open basin is not feasible.

The factors that will govern the decision on which type is preferable are basically the following:

Odour:

Overflow from combined sewers occurs when runoff exceeds sewer capacity. As such there is a time delay between when runoff commences and overflow occurs. This permits the initial surface runoff, which carries the most pollutants as well as deposits from previous low flow periods, to be conveyed to the sanitary trunk sewers and the sewage treatment plant. At the time C.S.O. begins, the heavier pollutants have passed the point of C.S.O., and the flow has become somewhat cleaner through further dilution. Although often ignored in the design of overflow regulators, it is relatively simple to incorporate scum baffles that will restrict floatables from discharging via C.S.O. It is recommended that as part of the revamping of the C.S.O. regulators, such scum baffles be installed.

Since the detention facilities are proposed to empty within 24 hours, odours from standing water will be minimal. The main concern will be odour due to settling of solids containing organics, particularly if such are allowed to accumulate over longer periods. With a closed tank this is more likely to occur than with an open basin. On the other hand, a closed tank will have mechanical ventilation, with discharge elevated above ground level, which should make the odour less objectionable.

If an open basin has a grassed lined bottom, organic settlement as well as heavy metals will act as fertilizer, increase grass growth and thus provide a natural removal of pollutants.

We foresee only minor odour problems, and on this aspect alone there may not be much difference between an open basin or an enclosed tank with mechanical ventilation.

Aesthetics:

It is difficult to make a dry pond aesthetically pleasing, particularly if it is to be concrete lined. An underground tank, however, will permit the site to be restored to its present use which is particularly important where the location is within an existing park. The detention facilities in the vicinity of Rockcliffe Boulevard, as presented in this report, are in this category, and assuming funds can be made available for the additional cost of underground tanks, such would be preferable.

The facility required adjacent to the Hyde Avenue tank is probably sufficiently removed from public view and an open basin in this location may be acceptable.

Safety:

Underground tanks will have access only through a small entrance building with a stairway leading down into the tank. Emergency overflows where required will have gratings prohibiting access.

Open basins, however, will require fencing with a minimum height of six feet to restrict unauthorized access to the site. Since the basins are normally dry, or with shallow water depth during short durations, fencing should provide adequate safety.

Maintenance:

Maintenance of detention facilities is dependent upon several factors.

Enclosed tanks will require infrequent removal of sediments. The existing tank at Hyde Avenue is cleaned six to eight times per year; each time with a four man crew over a period of two days. Cleaning consists of hosing the floor with water, and in some cases removal of sand and silt by buckets hoisted to the surface.

To facilitate cleaning of open basins, concrete lining of the bottom would be preferable. Only in the case of the basin at Hyde Avenue, where the existing underground tank will act as a sand trap and grit remover, can sedimentation be expected to be very minor, permitting a grass-lined basin.

Similar to other open space, grassed areas will need mowing from time to time, as well as spraying for weed control.

Cost:

As the cost estimates show, open basins cost considerably less than underground tanks to construct. However, open basins rule out any other use of the land. This may be acceptable for the site ajacent to the existing Hyde Avenue tank, but is likely not acceptable for the site west of Rockcliffe Boulevard which presently is a park, or the site east of Rockcliffe Boulevard which is being considered for industrial development.

2.1 Hyde Avenue

Required: Detention volume for 4,000 m^3 to 16,000 m^3 at 4,000 m^3 intervals

There is at present an underground detention tank at the termination of Hyde

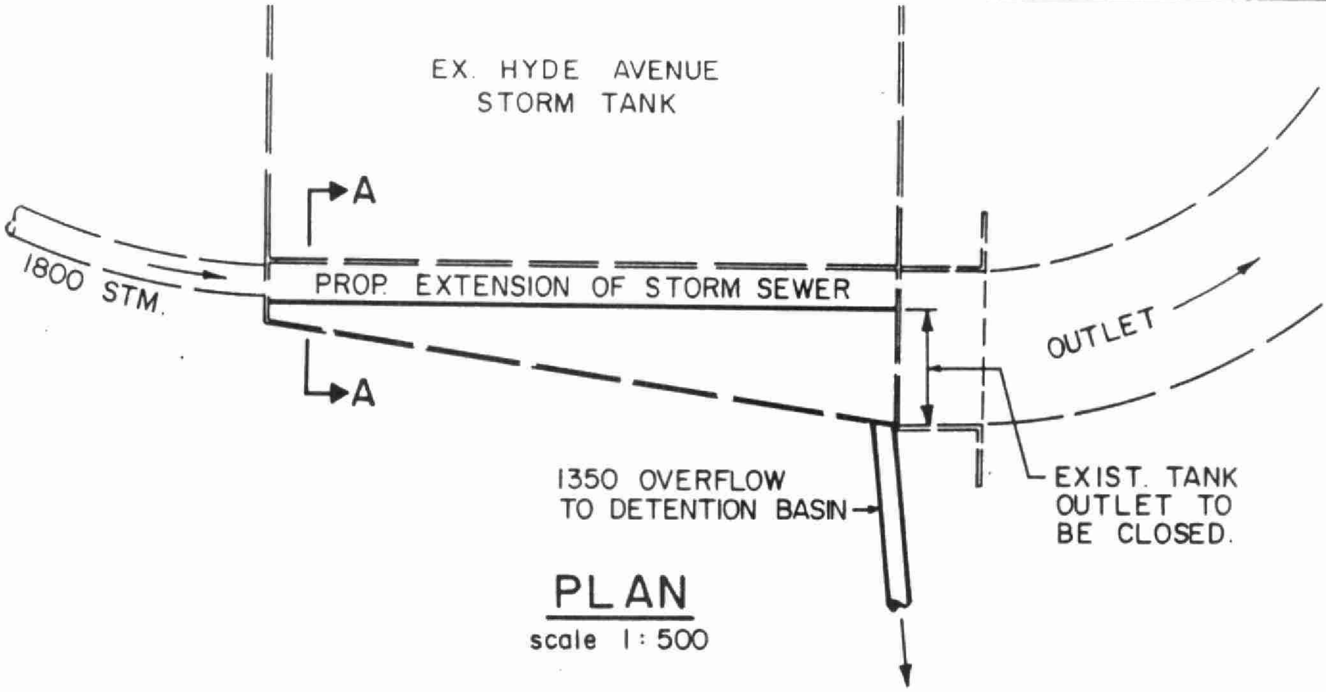
Avenue, adjacent to Black Creek. The open space immediately north of this tank (Plate 1-1) is ideally located for the additional detention required, making it possible to use the existing tank for sedimentation, thus permitting relatively cleaner overflow during less frequent runoff events to enter the new detention facility. Under such conditions, an open basin should be acceptable,particularly since it is not near residences, normally will be dry and only infrequently subject to overflow from the existing tank. Since grass is an excellent media for treatment of water polluted with heavy metals and organics, a grass lined basin may be preferable in this particular location.

In order to prevent the flow from the recently installed storm trunk sewer from Rotherham Avenue from entering the proposed detention facility, we propose to extend the storm sewer through the existing tank as shown schematically on Figure 1-2. Flow in excess of the 8,000 $m^3 \pm$ capacity of the present tank will be diverted into the proposed facility as indicated.

To prevent excess hydrostatic pressure on the roof of the existing tank during a rare event when the proposed detention facility is full, an emergency overflow from the tank is proposed, as shown on Figure 1.2. The modification to the existing tank will raise the outfall level from the present elevation 104.73 to 105.47, which is about 0.6 m below the 5 year floodline elevation as tentatively established by MTRCA. It may be possible to raise the outlet levels to the 5 year floodline by allowing some surcharge on the roof of the existing tank, but this will require a detailed structural analysis.

Cost Summary (as per Tables 1-2 and 1-3 in Section 5)

Volume	Basin		Tank
	Grass Lined	Concrete Bottom	
4,000 m^3	\$ 210,000	\$ 240,000	\$ 559,000.00
8,000 m^3	\$ 257,000	\$ 302,000	\$ 883,000.00
12,000 m^3	\$ 298,000	\$ 358,000	\$1,208,000.00
16,000 m^3	\$ 332,000	\$ 412,000	\$1,527,000.00



PLAN

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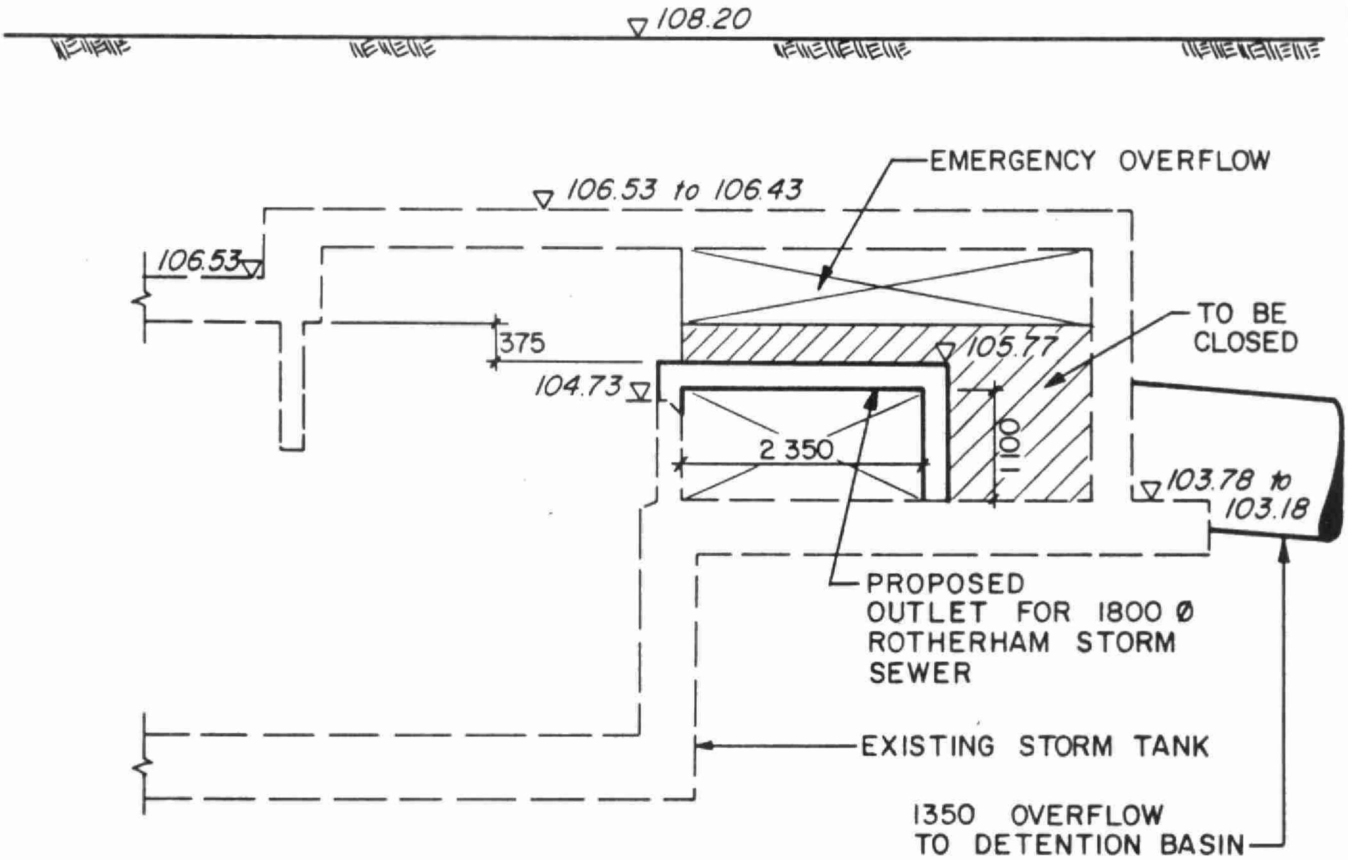


SECTION A-A

scale 1:75

EXTENSION OF ROTHERHAM STORM SEWER

figure 1-2

The possibility of conveying C.S.O from Hyde Avenue to a facility at Rockcliffe Boulevard serving the Site 3 regulator was investigated. A common facility would result in a reduction of the total detention volume between the Hyde Avenue and Rockcliffe Boulevard regulators of 8,000 m^3, as determined by the PCC 2 study. The volumes required for separate facilities are 16,000 m^3 for Hyde Avenue and 36,000 m^3 for Rockcliffe, serving Site 3, Mt. Dennis and Rockcliffe drainage areas, a total of 52,000 m^3, whereas conveyance of the excess flow from Hyde Avenue to a common faility at Rockcliffe will decrease the total volume to 44,000 m^3. This represents a potential saving of $570,000 based on underground tank, or $200,000 if an open basin is selected.

The cost of installing a pipe from Hyde Avenue to the Rockcliffe facility is estimated to cost about $1,360,000 if the facility is located east of Rockcliffe Boulevard,and $1,500,000 if located west of Rockcliffe Boulevard. As such it is not cost-efficient to eliminate the proposed facility at Hyde Avenue.

2.2 Rockcliffe Boulevard

Required: Detention volume for 4,000 m^3 to 36,000 m^3 at 4,000 m^3 intervals.

There are presently three combined sewer overflows to Black Creek in the vicinity of Rockcliffe Boulevard, as shown on Plate 1-2. The overflow from the Mt. Dennis drainage area and the Rockcliffe drainage area discharges to Black Creek channel west of Rockcliffe Boulevard, on the north side and south side respectively. The overflow from Site 3 is located about 150 metres east of Rockcliffe Boulevard.

The detention volumes required for each of the three drainage areas (assuming a separate facility at Hyde Avenue), as obtained from the PCC2 study are:

Mt. Dennis drainage area	16,000 m^3
Rockcliffe drainage area	4,400 m^3
Site 3 drainage area	15,600 m^3
Total	36,000 m^3

There are two sites that are technically suitable for location of a detention facility; the vacant lands owned by Metropolitan Toronto south of Black Creek and east of Rockcliffe Boulevard, and the open space between Black Creek and Alliance Avenue west of Rockcliffe Boulevard.

The use of a single detention facility to serve the three drainage areas will require an inverted siphon under Black Creek, which will result in ongoing maintenance to prevent severe silt accumulation in the siphon. By providing a small diameter drain from the low point of the siphon to the trunk sewer, the siphon can be kept dry, thus reducing the potential for gradual sedimentation during low flows. A single facility north of the channel serving the Mt. Dennis drainage area only, and another facility south of the channel serving the Rockcliffe drainage area and the C.S.O. from Site 3 will eliminate the need for a siphon.

The site west of Rockcliffe Boulevard is used as a park, which likely will rule out an open basin being acceptable to the adjacent residents.

The site east of Rockcliffe Boulevard, although vacant at the present, is being considered for an industrial subdivision. Since a decision on this has not been finally made, we have also investigated the possibility of using the site on the west side of Rockcliffe Boulevard for a single facility.

These possibilities have resulted in the following alternatives being investigated:

A. <u>East of Rockcliffe Boulevard</u>

1. Detention tank serving Rockcliffe and Site 3 (Plate 1-2).
2. Detention basin serving Rockcliffe and Site 3 (Plate 1-3).
3. Detention tank serving Rockcliffe, Site 3 and Mt. Dennis (Plate 1-4).
4. Detention basin serving Rockcliffe, Site 3 and Mt. Dennis (Plate 1-5).

B. West of Rockcliffe Boulevard

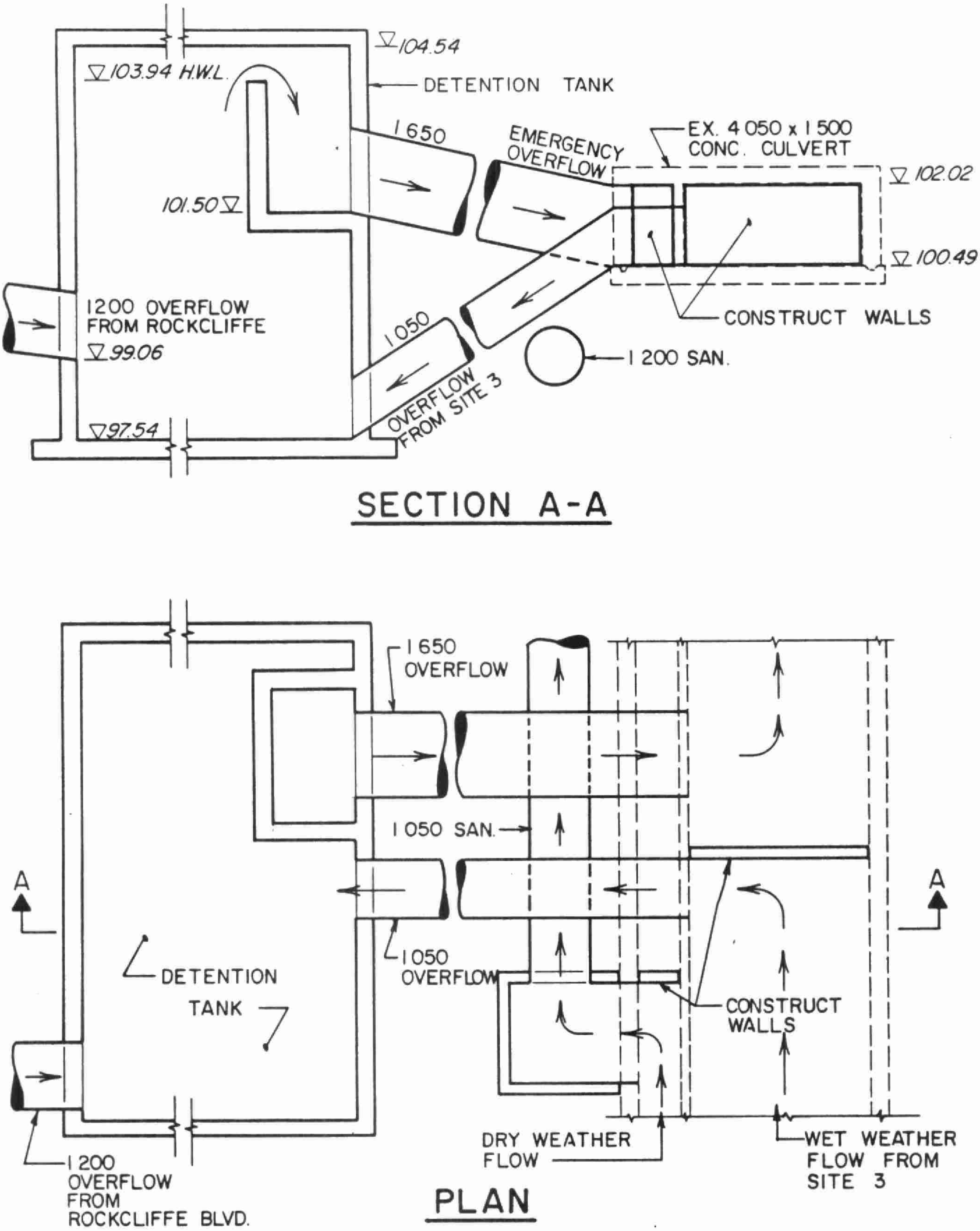
1. Detention tank serving Mt. Dennis area (Plate 1-6).
2. Detention basin serving Mt. Dennis area (Plate 1-7).
3. Detention tank serving Mt. Dennis, Rockcliffe and Site 3 (Plate 1-8).
4. Detention basin serving Mt. Dennis, Rockcliffe and Site 3 (Plate 1-9).

The site east of Rockclffe Boulevard is ideally located between the overflow pipe from Rockcliffe drainage area and the overflow culvert from Site 3, thus readily accommodating the flows from both areas with a minimum of piping. The site can also accommodate the C.S.O. from the Mt. Dennis Area through an inverted siphon under the Black Creek channel.

Outlet from this detention facility either as an open "dry" basin or an underground tank will be to the existing Metropolitan Toronto trunk sewer through a smaller diameter pipe with a static or adjustable flow control device. The elevations are such that operation will be by gravity and no pumping will be necessary.

Overflow from a detention facility in this location can be accommodated through the unused part of the existing culvert from Site 3 regulator, as shown schematically on Figure 1-3 for underground tanks and Figure 1-4 for open basins. The overflow elevation will be about 0.5 m above the 5 year floodline if a common facility for the three drainage areas is selected, or close to the 100 year floodline for a facility serving Rockcliffe and Site 3 drainage areas only.

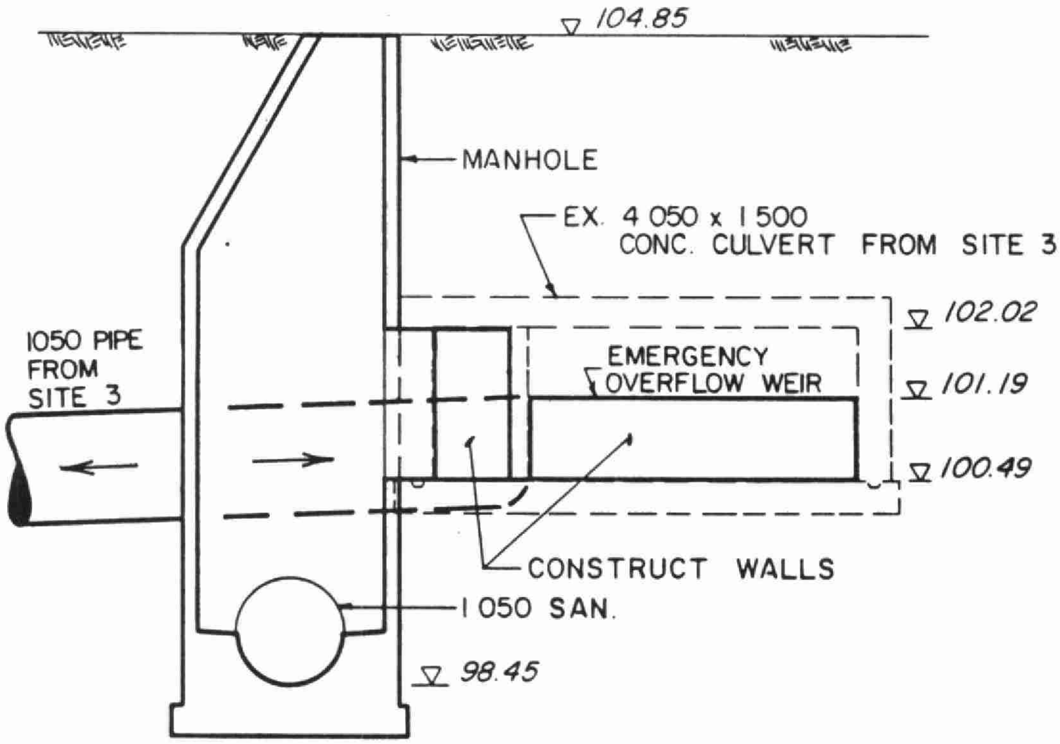
The site west of Rockcliffe Boulevard between Black Creek channel and Alliance Avenue is ideally located for the C.S.O. from the Mt. Dennis area. Flows from the two C.S.O. pipes on Rockcliffe Avenue can readily be diverted into the detention facility. Outlet to the Metropolitan trunk sewer can be provided by gravity through a flow control device with a pre-determined maximum flow rate. The overflow elevation will be about 1.5 m above the 5 year floodline.



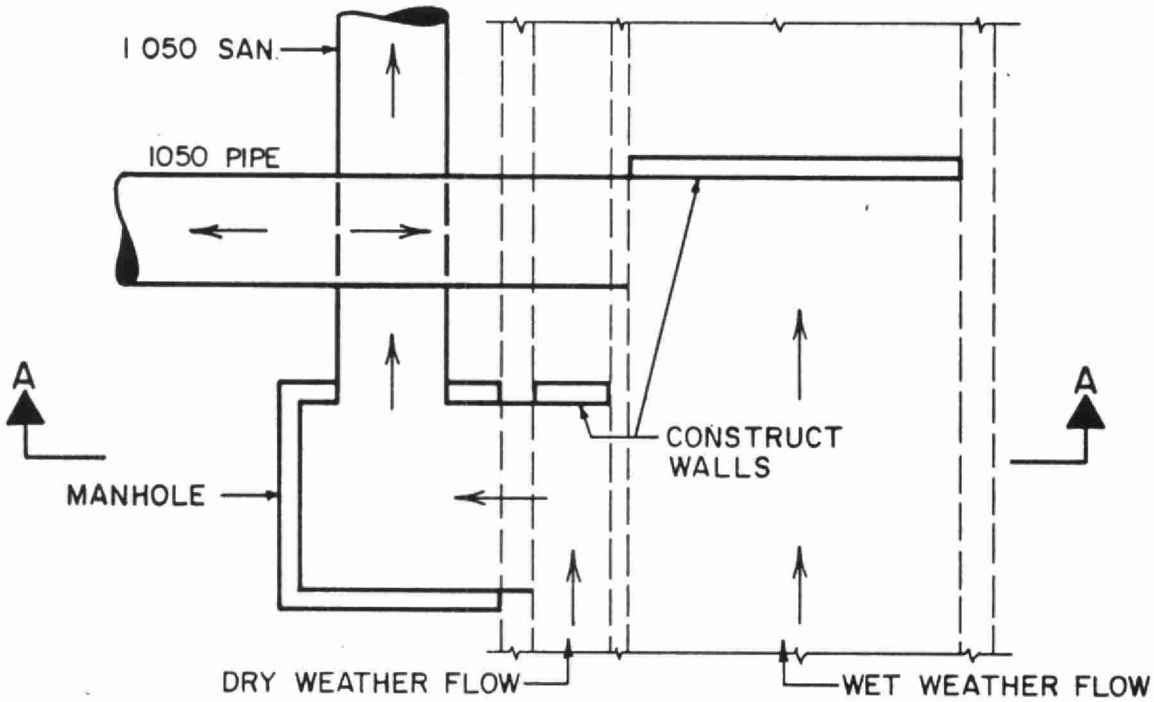
CONNECTIONS BETWEEN OVERFLOW CULVERT FROM SITE 3 REGULATOR & TANK EAST OF ROCKCLIFFE BLVD.

REFER TO PLATE 1-2

figure 1-3



SECTION A-A



PLAN

CONNECTIONS BETWEEN OVERFLOW CULVERT FROM SITE 3 REGULATOR & BASIN EAST OF ROCKCLIFFE BLVD.

REFER TO PLATES 1-4 and 1-5

figure 1-4

The site can also serve as detention for the C.S.O. from Site 3 and Rockcliffe drainage area, utilizing an inverted siphon under Black Creek as shown on Plates 1-8 and 1-9. The same comments made above for a siphon to the east site apply. The overflow elevations will be about 1.5 m above the 5 year floodline with or without connection to the overflow from Site 3.

Cost Summary

In order of overall cost, including all associated piping, as summarized in Table 1-3 to 1-9 inclusive, the following alternatives are available for storing C.S.O. from the three drainage areas, presented in increments of 4,000 m^3. These costs are also presented graphically in Figure 1-5.

A. East of Rockcliffe Boulevard

1. Serving Rockcliffe plus Site 3 drainage areas

Volume	Basin		Tank
	Grass Lined	Concrete Bottom	
4,000 m^3	$ 303,000	$ 323,000	$ 613,000
8,000 m^3	$ 406,000	$ 444,000	$ 890,000
12,000 m^3	$ 508,000	$ 564,000	$1,169,000
16,000 m^3	$ 610,000	$ 684,000	$1,448,000
20,000 m^3	$ 712,000	$ 804,000	$1,727,000

2. Serving Mt. Dennis plus Rockcliffe plus Site 3 drainage areas

Volume	Basin		Tank
	Grass Lined	Concrete Bottom	
4,000 m^3	$ 412,000	$ 447,000	$ 766,000
8,000 m^3	$ 509,000	$ 574,000	$1,070,000
12,000 m^3	$ 606,000	$ 701,000	$1,374,000
16,000 m^3	$ 703,000	$ 828,000	$1,678,000
20,000 m^3	$ 800,000	$ 955,000	$1,982,000
24,000 m^3	$ 897,000	$1,087,000	$2,286,000
28,000 m^3	$ 999,000	$1,209,000	$2,592,000
32,000 m^3	$1,091,000	$1,336,000	$2,866,000
36,000 m3	$1,188,000	$1,463,000	$3,161,000

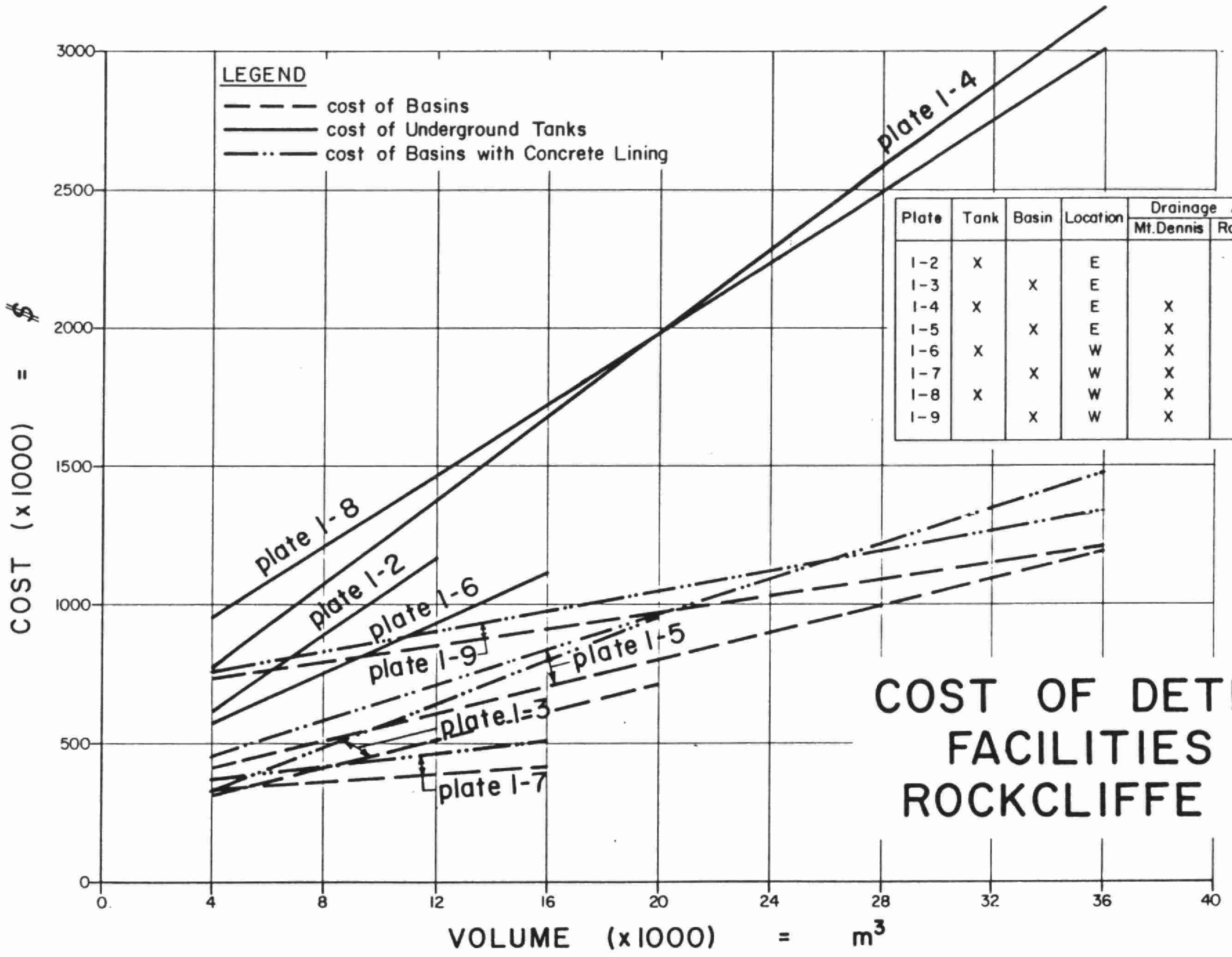


Plate	Tank	Basin	Location	Drainage Areas Served		
				Mt.Dennis	Rockcliffe	Site 3
I-2	X		E		X	X
I-3		X	E		X	X
I-4	X		E	X	X	X
I-5		X	E	X	X	X
I-6	X		W	X		
I-7		X	W	X		
I-8	X		W	X	X	X
I-9		X	W	X	X	X

COST OF DETENTION FACILITIES AT ROCKCLIFFE BLVD.

figure I-5

B. West of Rockcliffe Boulevard

1. Serving Mt. Dennis drainage area

Volume	Basin Grass Lined	Basin Concrete Bottom	Tank
4,000 m^3	$ 337,000	$ 362,000	$ 571,000
8,000 m^3	$ 356,000	$ 400,000	$ 757,000
12,000 m^3	$ 386,000	$ 449,000	$ $933,000
16,000 m^3	$ 416,000	$ 498,000	$1,115,000

2. Serving Mt. Dennis plus Rockcliffe plus Site 3 drainage areas

Volume	Basin Grass Lined	Basin Concrete Bottom	Tank
4,000 m^3	$ 728,000	$ 748,000	$ 953,000
8,000 m^3	$ 788,000	$ 827,000	$1,210,000
12,000 m^3	$ 848,000	$ 906,000	$1,460,000
16,000 m^3	$ 908,000	$ 985,000	$1,720,000
20,000 m^3	$ 968,000	$1,064,000	$1,980,000
24,000 m^3	$1,028,000	$1,143,000	$2,240,000
28,000 m^3	$1,088,000	$1,223,000	$2,495,000
32,000 m^3	$1,148,000	$1,302,000	$2,750,000
36,000 m^3	$1,208,000	1,383,000	$3,010,000

2.3 Berry Road

Required: Detention volume of 4,200 m^3

The present overflow regulator on Stephen Drive just north of Berry Road diverts flows exceeding 83 percent of pipe diameter to an outfall pipe discharging to Humber River. There is not sufficient space available for a detention facility in the immediate vicinity of the regulator. It is possible, however, to locate a detention tank in the open space east of Stephen Drive about 300 m south of Berry Road, as shown on Plate 1-10.

Due to the severe topography, an open basin is considered impractical. The close proximity to residences may also make acceptance of an open basin doubtful. An existing storm sewer will need to be routed around the tank. The elevations are such that the tank can operate by gravity without the need for pumping. The overflow from the tank to the trunk sewer will be provided with a check valve to prevent inflow during times of normal flow in the trunk sewer. This arrangement is shown schematcially in Figure 1-6. The overflow will be above the floodline for the Regional storm.

Cost Summary

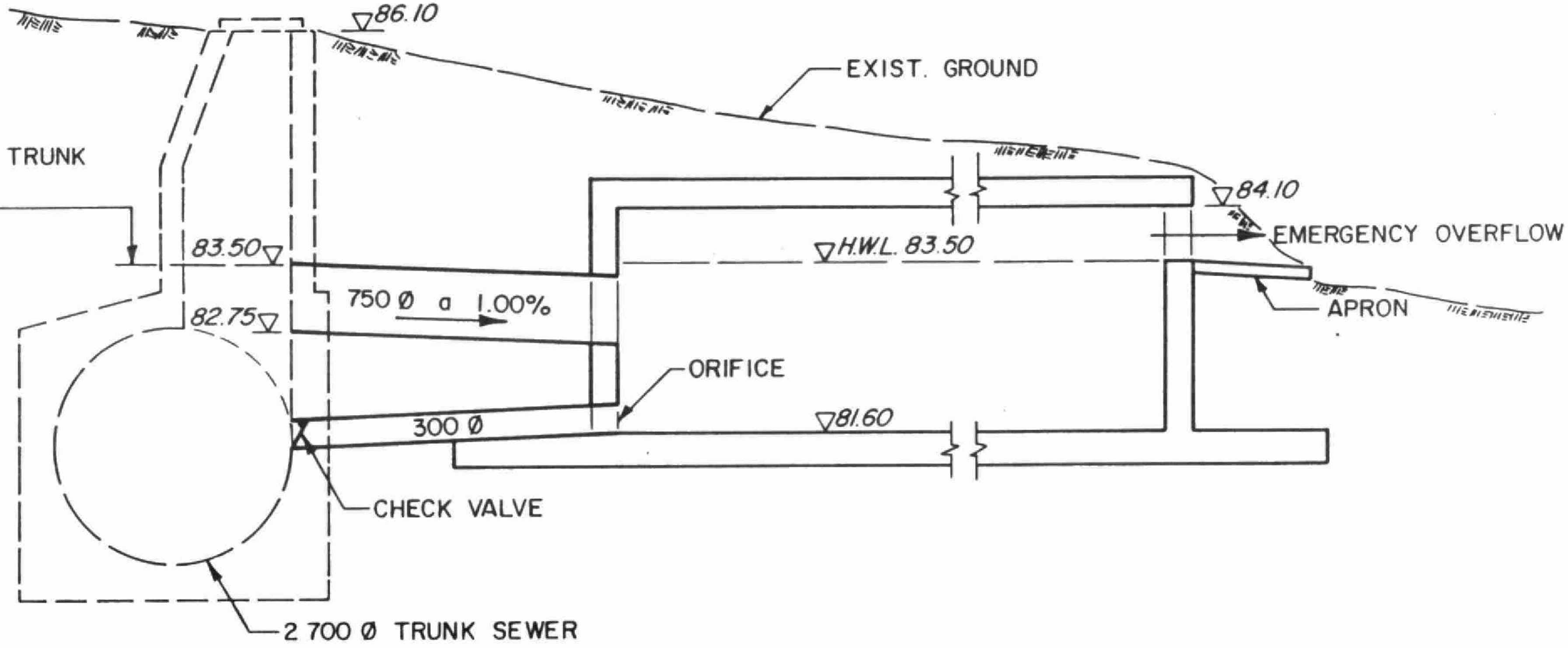
4,200 m^3 Detention tank in Park (Table 1-11)	$	621,000.00

It is also possible to provide a detention basin in the Humber River flood plain, adjacent to the present C.S.O. outfall, as shown on Plate 1-11. In order to prevent siltation during times when the water level rises in the Humber River, we propose that berms be constructed above the existing grade. Due to the nature of the material on the site, consisting of grey fine sandy silt with some organic traces, we propose side slopes of about 1:4. This avoids importing fill, but results in more earthwork on the site. The top elevation of the berms as proposed will be above the 100 year floodline.

In order to remove storm water from the present overflow sewer, it will be necessary to extend the storm sewer from Stephen Drive along side the overflow sewer to an outlet bypassing the detention basin. The basin will require pumping in order to drain. The cost of the pump and forcemain installation as well as rerouting of the storm sewer from Stephen Drive is included in the cost estimate.

Cost Summary

4,200 m^3 Detention Basin in Humber Valley including pumps, forcemain etc. (Table 1-12)	$	513,000.00
Add for concrete lining, if required	$	41,000.00



CONNECTIONS BETWEEN HUMBER SANITARY TRUNK SEWER & TANK EAST OF STEPHEN DRIVE

REFER TO PLATE 1-10

2.4 Humber Water Pollution Control Plant

Required: Detention volume of 4,200 m^3

As shown on Plate 1-12, there is presently unused space north of the entrance to the Humber Water Pollution Control Plant, which with some re-grading of the adjacent berm is sufficient to accommodate an open detention basin.

Since the Humber trunk sewer normally flows part full, it is not practical to provide gravity drains for the lower 0.5 metre of the basin. As such, we therefore propose a submersible pump where the effluent line enters the intake building, as shown schematically on Plate 1-12.

Cost Summary

4,200 m^3 Detention Basin at Humber W.P.C.P.	$ 309,000.00

Concrete lining included.

3 - DUPLICATION OF EXISTING TRUNK SEWER

3.1 Black Creek Sanitary Relief Sewer from Site 3 Regulator to Humber Sanitary Trunk Sewer

Required: 2.75 m^3/sec additional capacity

The feasibility of providing additional sewer capacity from Site 3 regulator to the Humber sanitary trunk sewer was investigated. The total capacity required is 5.82 m^3/sec. Assuming a pipe friction factor n = 0.013, this will require a 900 mm sewer installed generally parallel with the existing 1650 mm Metropolitan Toronto trunk sewer, as shown on Plates 1-13 and 1-14.

The estimated cost of this sewer is $1,445,000. We assume the sewer can be located within the existing easements, and as such have made no allowance for any easement costs. However, due to the difficulties experienced when the existing two sewers were installed, we have allowed for tight sheeting throughout, and resultant reduction in daily production. We have also recognized the need for greater requirements for ground restoration of the section through the Lambton golf course, which is reflected in the estimated cost.

4 - OPERATION AND MAINTENANCE COSTS

4.1 Cleaning of Tanks

Discussions were held with City of York Works Yard staff to determine the manpower requirements to clean the existing Hyde Avenue tank and to estimate the manpower requirements for maintaining the proposed facilities. Based on the proposed designs discussed with City staff, it was estimated that a 10,000 m^3 facility would require approximately 15 man days per year to maintain. This represents an annual cost of about $3,000. The present value costs (or capital costs) were converted from annual costs using a 70 year time period and a net interest rate (interest rate minus inflation rate) of 7 percent.

Assuming this cost is generally proportioned to the volume, the following annual and present value costs for cleaning can be assumed for tanks:

VOLUME	ESTIMATED ANNUAL COST	PRESENT VALUE COST
4,000 m^3	$ 2,000	$ 28,000
8,000 m^3	2,400	34,000
12,000 m^3	3,600	$50,000
16,000 m^3	4,800	68,000
20,000 m^3	6,000	84,000
24,000 m^3	7,20	100,000
28,000 m^3	8,400	118,000
32,000 m^3	9,600	136,000
36,000 m^3	10,800	152,000

4.2 Cleaning of Open Basins

If an open basin is selected for the detention facility adjacent to the existing Hyde Avenue tank, most of the sediments will settle in the tank, resulting in only minor sediments in the open basin. If the basin is to have an impermeable bottom, such as concrete lining, cleaning can effectively be done by hosing with water. We estimate six man days yearly as being sufficient, equal to a cost of about $1,200 annually, or a present value cost of $17,000.

The open basin proposed at the Humber W.P.C.P. will be subject to larger volumes of sediment and as such a concrete invert has been allowed for in the cost estimate. Cleaning can then readily be carried out manually by the staff of the W.P.C.P. Since it might be possible to schedule this operation during periods when staff may not otherwise be engaged, we estimate the additional cost will be minimal, say $1,000 annually or $14,000 on a present value basis.

In the event open basins are selected for any of the other locations, concrete lining of the bottom and lower part of the side slope will greatly facilitate cleaning. Unfortunately, this will increase the flow of storm water to the sanitary sewer during rainfall events, due to direct runoff from concrete surfaces. The additional cost of concrete lining of bottom and about one metre of lower part of the side slope is stated in the cost estimates presented in Section 5.

VOLUME	ESTIMATED ANNUAL COST	PRESENT VALUE COST
4,000 m^3	$ 1,000	$ 14,000
8,000 m^3	1,000	14,000
12,000 m^3	1,200	17,000
16,000 m^3	1,600	22,000
20,000 m^3	2,000	28,000
24,000 m^3	2,400	34,000
28,000 m^3	2,800	39,000
32,000 m^3	3,200	45,000
36,000 m^3	3,600	50,000

4.3 Instrumentation

In the cost estimates are allowances for supply and installation of the following instrumentation:

a) Remote control of opening and closing of drain.

	Capital Cost	Annual Cost *
This can be achieved with a motorized sluice gate, installed in a chamber with manually operated handwheel and connection to hydro and low voltage transmitter.		
Per installation	$ 15,000	$ 1,000

b) Flow measurement of inflow and outflow

Flow measurements for an installation of this type can be made by using a pressure transducer mounted in invert of pipe and connected by cable to a transmitter with output to a remote recorder.		
Estimated cost per installation	$ 6,000	$ 1,500

c) Measurement of depth of storage

Depth of storage in either open basins or underground tanks can be measured by an ultrasonic level sensor mounted above high water level and connected to a transmitter with output to a remote recorder.		
Estimated cost per installation	$ 6,600	$ 1,500

d) Remote Recorder

Data logger with printer		
- For detention facilities at Hyde Avenue and Ratcliffe Boulevard	$ 10,000	$ 2,000
- For detention facility at Humber WPCP, Stephen Road or at Berry Road outfall.	$ 5,000	$ 1,000

* Average Annual Maintenance including replacements. These costs have been converted to present value costs and included in the cost summary for operation and maintenance.

4.4 Flow regulator

As a regulator for the outflow to the sanitary trunk sewers, an orifice can be used with an opening designed on the basis of head and desired flow rate. In order to reduce the risk of clogging, it will be preferable to use a "Hydro-Brake", which also is a static device, but with a much larger opening than an orifice for passage of the same flow. The cost of Hydro-Brakes has been allowed for in the estimates.

5 - ESTIMATE OF COSTS

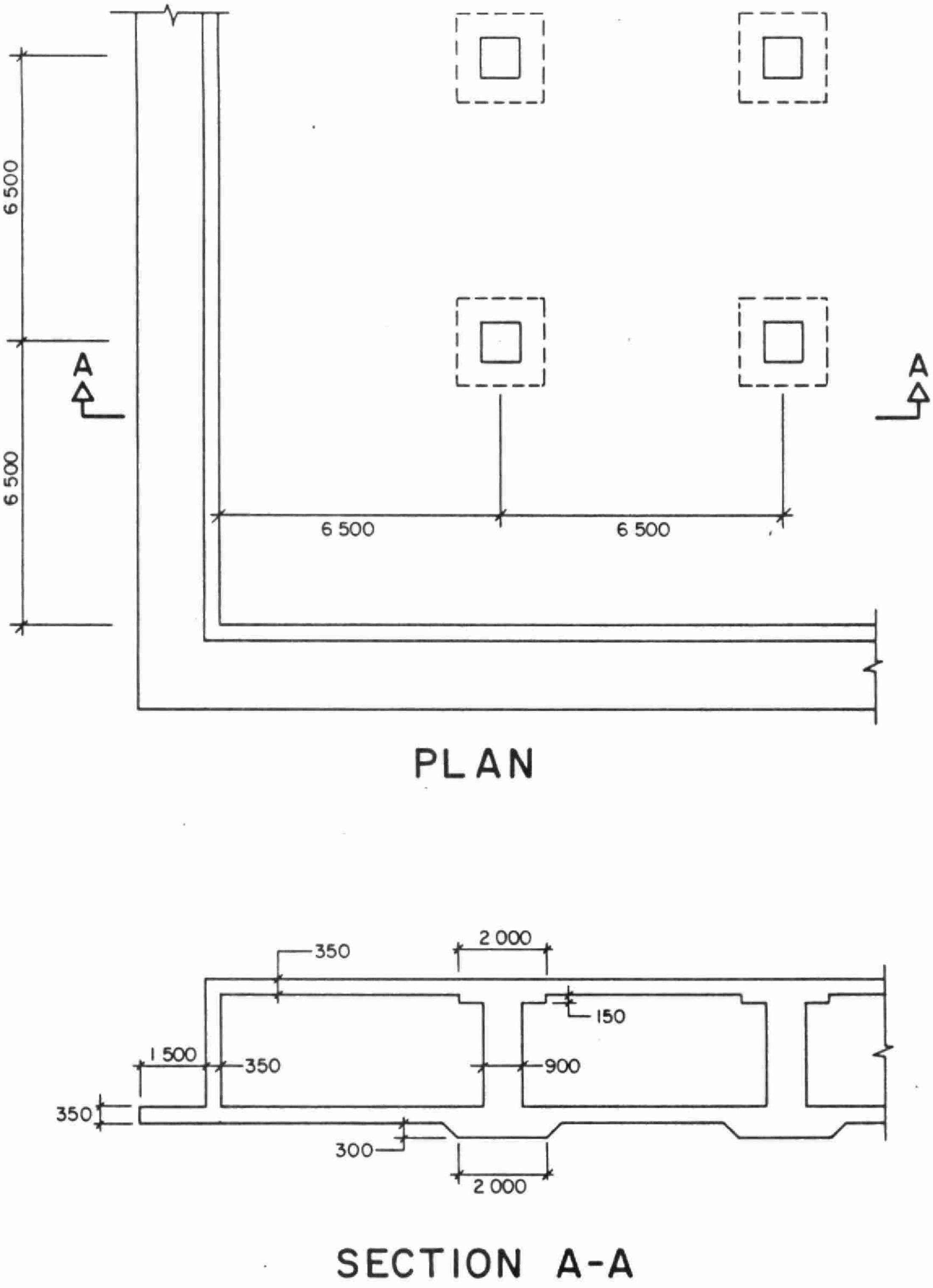
Methodology

In order to prepare a cost estimate for the underground detention tanks, a structural design was made to determine the basic dimensions of the structural components. These are shown on Figure 1-7.

Earth excavations for each detention facility were calculated on the basis of the most recent topographic maps and the proposed dimensions and elevations as shown on the drawings herein. Calculations of earth excavations for sewers and for underground tanks are based on the cut lines shown in Figure 1-8. Surplus excavated material has been assumed to be removed by trucking. Major earthwork is assumed to be carried out between June and September.

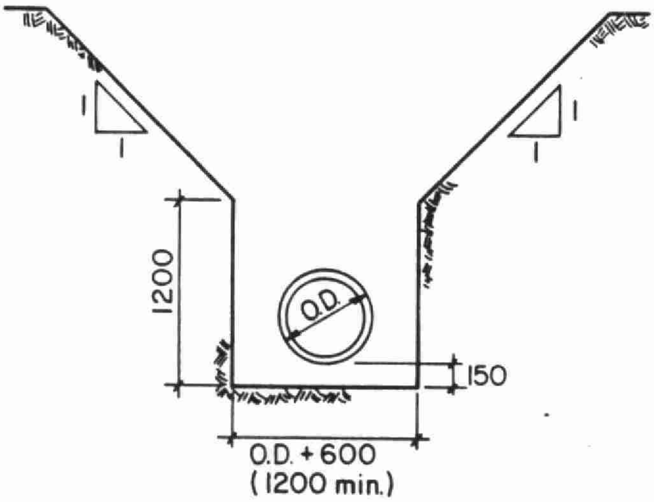
In addition, allowances have been made for:

- restoration of areas affected by construction
- chain link fencing around open basins
- underdrains and granular base for open basins
- inlet and outlet piping
- overflows to Black Creek or Humber River
- access roads
- entrance building and stairways to underground tanks
- instrumentation (see "Operation and Maintenance Costs")
- ventilation of underground tanks
- hydro and water supply
- extension of Rotherham Avenue storm sewer outfall
- engineering and contingencies 20 percent

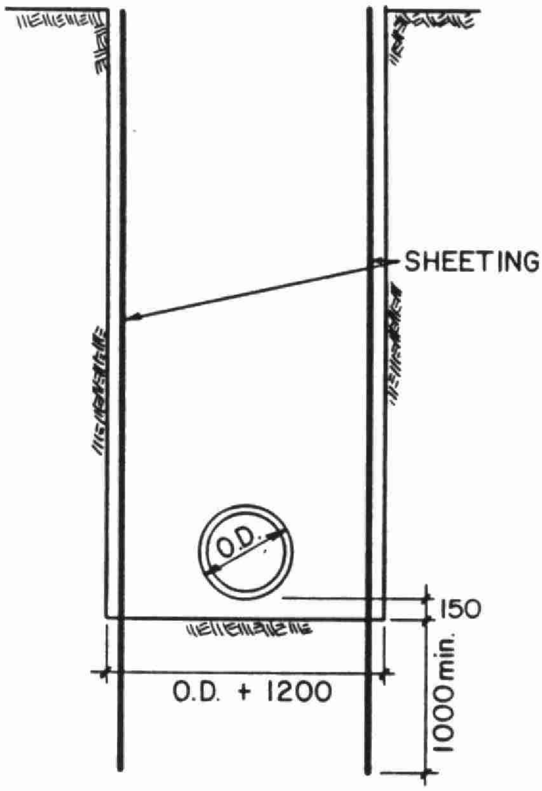


BASIC DIMENSIONS FOR UNDERGROUND TANKS
(for cost estimating only)

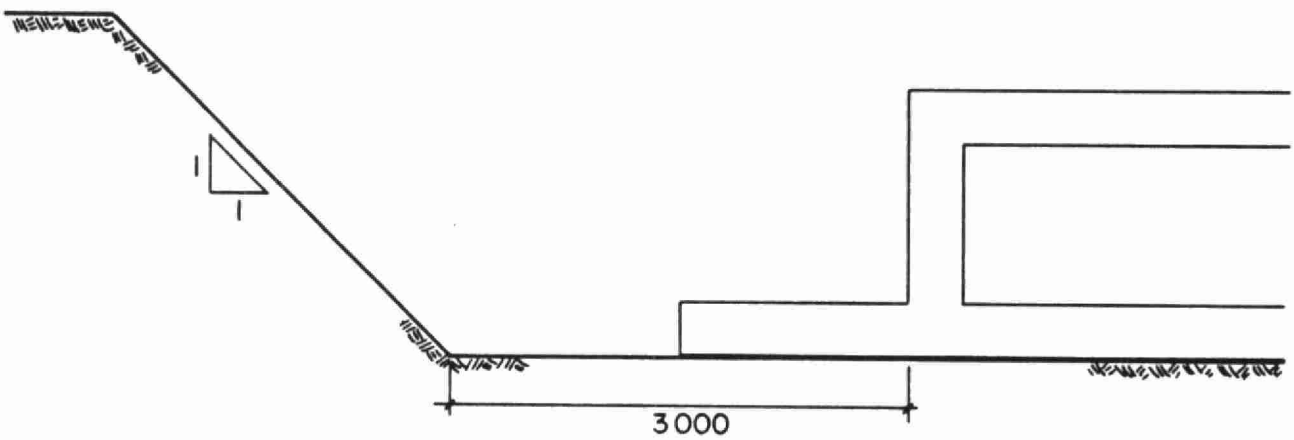
figure I-7



SEWERS IN
OPEN CUT



SEWERS WITH
TIGHT SHEETING



UNDERGROUND TANKS

CUT LINES FOR CALCULATION OF EARTH EXCAVATION

figure 1-8

Estimated costs of sewers include for:

- excavation and backfilling
- removal of surplus excavation
- bedding
- concrete pipe of classification suitable for design depths
- manholes
- restoration of working areas
- engineering and contingencies (twenty percent)

A breakdown of the major cost items are presented in Tables 1-1 to 1-14.

The cost to treat, at the Humber Water Pollution Control Plant, the combined sewer overflows stored in the proposed facilities was established. Two unit costs for treatment were established by the Metropolitan Works Department (letter from Mr. R. M. Pickett, August 1, 1985). The unit cost selected for this study ($0.045/$m^3$ treated) is based on incremental costs incurred by the treatment plant when treating flows in excess of the daily average flows. Based on the 1979 precipitation data, if all the combined sewer overflows (excluding the July 11, 1979, event which had a return period in excess of 3 years) were stored, the total volume treated would be about 330,000 m^3. This is equivalent to approximately 0.1 percent of the annual volume of sewage treated. The annual cost to provide treatment would, therefore, be roughly $14,800. The present value cost of treatment (over a 70 year period) would be about $207,000.

TABLE 1-1

HYDE AVENUE DETENTION BASIN
Plate 1-1

Item	Volume			
	4,000 m^3	8,000 m^3	12,000 m^3	16,000 m^3
Excavation	14,200 m^3	20,500 m^3	28,300 m^3	34,300 m^3
Cost	\$ 2.00/m^3	\$ 2.00/m^3	\$ 2.00/m^3	\$ 2.00/m^3
Subtotal	\$ 28,400	\$ 41,000	\$ 56,600	\$ 68,600
Underdrains	\$ 3,000	\$ 4,500	\$ 6,000	\$ 8,000
Sodding	7,500 m^2	15,400 m^2	20,500 m^2	25,000 m^2
Cost	\$ 3.00/m^2	\$ 3.00/m^2	\$ 3.00/m^2	\$ 3.00/m^2
Subtotal	\$ 22,500	\$ 46,200	\$ 61,500	\$ 75,000
Fencing	180 m	240 m	280 m	320 m
Cost	\$28.00/m	\$28.00/m	\$28.00/m	\$28.00/m
Subtotal	\$ 5,040	\$ 6,720	\$ 7,840	\$ 8,960
Piping, etc. (see below)	\$116,424	\$116,424	\$116,424	\$116,424
Total	\$175,364	\$214,844	\$248,364	\$276,984
Eng. & Cont. (20%)	\$ 35,636	\$ 43,156	\$ 49,636	\$ 55,016
Total (rounded)	\$210,000	\$257,000	\$298,000	\$332,000
Concrete lining (if required)	\$ 30,000	\$45,000	\$60,000	\$ 80,000
Total Capital Cost	\$240,000	\$302,000	\$358,000	\$412,000
Present Value Cost for Cleaning and Maintenance (70 years)	\$ 84,000	\$ 84,000	\$ 87,000	\$ 92,000
GRAND TOTAL	\$324,000	\$386,000	\$445,000	\$504,000

...cont..

TABLE 1-1 - cont..

PIPING (included above)

900 mm diameter overflow	22 m @ $233/m	$ 5,126
Overflow structure		7,000
1950 mm diameter influent	22 m @ $859/m	18,898
Headwall		14,500
Connection to existing structure		2,300
Extension of Rotherham Avenue Outfall (Fig. 1-2)		56,000
300 mm diameter outlet to trunk sanitary sewer		4,600
New MH on existing 1350 mm diameter trunk sewer		8,000
		$116,424

Add for concrete lining, if required:

4,000 m^3 Basin:	$30,000	Total: $240,000
8,000 m^3 Basin:	$45,000	Total: $302,000
12,000 m^3 Basin:	$60,000	Total: $358,000
16,000 m^3 Basin:	$80,000	Total: $412,000

TABLE 1-2

HYDE AVENUE DETENTION TANK

Item	Volume			
	4,000 m^3	8,000 m^3	12,000 m^3	16,000 m^3
Piping	$116,424	$116,424	$116,424	$116,424
Excavation & backfill	5,090	10,180	15,270	20,360
Excavation & disposal	6,600	13,200	19,800	26,400
Reinforced Concrete on forms	564 m^3 @ 380 = $214,320	1026m^3 @ 365 = $374,417	1496m^3 @ 350 = $331,151	1986m^3 @ 350 = $694,992
Reinforced Concrete in grade	414 m^3 @ 220 = $ 91,102	824 m^3 @ 210 = $172,872	1239m^3 @ 205 = $253,995	1666m^3 @ 200 = $333,270
Restoration	4000 m^2@3.00 $ 12,000	6000 m^2@3.00 $ 18,000	8000 m^2@3.00 $ 24,000	10000m^2@3.00 $ 30,000
Miscellaneous	$ 20,000	$ 30,000	$ 42,000	$ 52,000
Subtotal	$465,536	$736,093	$1,007,640	$1,273,446
Eng. & Cont. (20%)	$ 93,464	$147,907	$ 201,360	$ 254,554
Total Capital Cost	$559,000	$883,000	$1,208,000	$1,527,000
Present Value Cost for Cleaning and Maintenance (70 years)	$ 84,000	$104,000	$120,000	$138,000
GRAND TOTAL	$643,000	$987,000	$1,328,000	$1,665,000

TABLE 1-3

12,000 m^3 DETENTION TANK, EAST OF ROCKCLIFFE BOULEVARD (PLATE 1-2)

Excavation and backfill		2,300 m^3 @ 3.00	$ 6,900
Excavation and disposal		14,600 m^3 @ 5.50	$ 80,300
Reinforced concrete in forms		1,417 m^3 @ 350.00	$495,880
Reinforced concrete on grade		940 m^3 @ 200.00	$188,000
450 mm dia.outlet pipe to Metro Trunk		55 m @ 300.00	$ 16,500
Piping west of trunk:	1220 mm dia.	81 m @ 667.00	$ 54,027
	Manhole	1 @ 8000.000	$ 8,000
Piping east of trunk:	1050 mm dia.	20 m @ 495.00	$ 9,900
	1500 mm dia.	20 m @ 750.00	$ 15,000
Connections to existing culvert(2)			$ 2,400
1050 mm sanitary sewer		75 m @ 550.00	$ 41,250
Manhole for sanitary sewer		1 @ 7500.00	$ 7,500
Restoration		7,600 m^2 @ 3.00	$ 22,800
Miscellaneous: access road, entrance building, stairways, instrumentation, ventilation, hydro, water supply etc.			$ 25,000
Sub-total			$973,457
Engineering and contingencies 20%			$195,543
TOTAL CAPITAL COST			$1,169,000
Present Value Cost for Operation and Maintenance			$ 120,000
GRAND TOTAL			$1,289,000

TABLE 1-4

12,000 m^3 DETENTION BASIN, EAST OF ROCKCLIFFE BOULEVARD (PLATE 1-3)

Topsoil stripping	1,800 m^3 @ 1.75	$ 3,150
Excavation and disposal	35,500 m^3 @ 5.50	$195,250
Underdrains		$ 4,000
Sodding	8,299 m^2 @ 3.15	$ 25,853
Fencing	303 m @ 28.00	$ 8,480
1050 mm dia. sewer	27.5 m @ 343.00	$ 9,433
1200 mm dia. sewer	42.7 m @ 387.00	$ 16,525
1650 mm dia. sewer	40.0 m @ 600.00	$ 24,000
Inlets: 1050 mm dia.		$ 11,000
1200 mm dia.		$ 12,000
1650 mm dia.		$ 15,000
Connection to existing box culvert(2)		$ 16,000
450 mm dia. outlet to Metro.trunk	65 m @ 300.00	$ 19,500
1050 dia. mm sanitary sewer	75 m @ 550.00	$ 41,250
Manhole for sanitary sewer		$ 7,500
Miscellaneous: instrumentation, hydro and water supply etc.		$ 15,000
Sub-total		$423,941
Engineering and contingencies 20%		$ 85,059
TOTAL CAPITAL COST		$508,000
Add for concrete lining, if required		$ 56,000
		$564,000
Present Value Cost for Operation and Maintenance		$ 87,000
GRAND TOTAL		$651,000

TABLE 1-5

28,000 m^3 DETENTION TANK, EAST OF ROCKCLIFFE BOULEVARD (PLATE 1-4)

Excavation and backfill	29,000 m^3	@ 3.00	$ 87,000
Excavation and disposal	30,000 m^3	@ 5.50	$ 165,000
Reinforced concrete in forms	3,100 m^3	@ 350.00	$1,085,000
Reinforced concrete on grade	2,700 m^3	@ 200.00	$ 540,000
450 mm dia.sewer north side	60 m	@ 160.00	$ 9,600
900 mm dia.sewer north side	72 m	@ 300.00	$ 21,600
1200 mm dia. sewer north side	50 m	@ 400.00	$ 20,000
1500 mm dia. sewer north side	82 m	@ 430.00	$ 35,260
Manholes north side	2	@ 8000.00	$ 16,000
	3	@ 7000.00	$ 21,000
1500 mm dia. sewer west side	30 m	@ 700.00	$ 21,000
Manhole west side	1	@ 8000.00	$ 8,000
1050 mm dia.san.sewer east side	90 m	@ 350.00	$ 31,500
Manhole east side	1	@ 15000.00	$ 15,000
Restoration	18,000 m^2	@ 3.00	$ 54,000
Miscellaneous: access road, entrance building, stairways, instrumentation, ventilation, hydro, water supply etc.			$ 30,000
Sub-total			2,159,960
Engineering and contingencies 20%			$ 432,040
TOTAL CAPITAL COST			$2,592,000
Present Value Cost for Operation and Maintenance			$ 188,000
GRAND TOTAL			$2,780,000

TABLE 1-6

28,000 m^3 DETENTION BASIN, EAST OF ROCKCLIFFE BOULEVARD (PLATE 1-5)

Topsoil stripping & stockpile	4,100 m^3	@ 1.75	$ 7,175
Excavation and backfill	1,300 m^3	@ 3.00	$ 3,900
Excavation and disposal	81,000 m^3	@ 5.50	$445,500
Underdrains			$ 15,000
Sodding	27,000 m^2	@ 3.00	$ 81,000
Fencing	500 m	@ 28.00	$ 14,000
Inlets: 1200 mm dia.			$ 12,000
1050 mm dia.			$ 11,000
1500 mm dia.			$ 14,000
Outlet for 450 mm dia. pipe			$ 1,500
450 mm dia.sewer north side	71 m	@ 160.00	$ 11,360
1500 mm dia. sewer north side	106 m	@ 425.00	$ 45,050
900 mm dia.sewer north side	25 m	@ 300.00	$ 7,500
1200 mm dia. sewer north side	50 m	@ 400.00	$ 20,000
Manholes north side	2	@ 8000.00	$ 16,000
	3	@ 7000.00	$ 21,000
1200 mm dia. sewer west side	50 m	@ 675.00	$ 33,750
Manhole west side	1	@ 8000.00	$ 8,000
1050 mm dia.san.sewer east side	102 m	@ 330.00	$ 33,660
Manhole east side	1	@ 12000.00	$ 12,000
Miscellaneous: instrumentation, ventilation, hydro,etc.			$ 15,000
Sub-total			$828,395
Engineering and contingencies 20%			$165,605
TOTAL CAPITAL COST			$994,000
Add for concrete lining, if required			$215,000
			$1,209,000
Present Value Cost for Operation and Maintenance			$ 109,000
GRAND TOTAL			$1,318,000

TABLE 1-7

16,000 m^3 DETENTION TANK, WEST OF ROCKCLIFFE BOULEVARD (PLATE 1-6)

Excavation and backfill	3,700 m^3 @ 3.00	$	11,100
Excavation and disposal	16,000 m^3 @ 5.50	$	88,000
Reinforced concrete in forms	1,600 m^3 @ 350.00	$	581,000
Reinforced concrete on grade	325 m^3 @ 200.00	$	65,000
1200 mm dia. sewer north side	34 m @ 670.00	$	22,780
Manholes north side	2 @ 7000.00	$	14,000
450 mm dia.sewer south side	53 m @ 1500.00	$	79,500
Manhole south side	1 @ 8000.00	$	8,000
Restoration	10,200 m^2 @ 3.00	$	30,600
Miscellaneous: access road, entrance building, stairways, instrumentation, ventilation, hydro and water supply etc.		$	30,000
Sub-total		$	929,980
Engineering and contingencies 20%		$	185,020
TOTAL CAPITAL COST			$1,115,000
Present Value Cost for Operation and Maintenance		$	109,000
GRAND TOTAL			$1,224,000

TABLE 1-8

16,000 m^3 DETENTION BASIN, WEST OF ROCKCLIFFE BOULEVARD (PLATE 1-7)

Topsoil stripping & stockpile	2,000 m^3	@ 1.75	$ 3,500
Excavation and disposal	27,500 m^3	@ 5.50	151,250
Underdrains			$ 10,000
Sodding	11,300 m^2	@ 3.00	$ 33,900
Fencing	400 m	@ 28.00	$ 11,200
Inlets for 1200 mm dia. pipe			$ 13,000
Manholes	2	@ 8000.00	$ 16,000
Manhole on Metro.trunk	1	@ 8000.00	$ 8,000
450 mm dia. sewer south side	53 m	@ 1500.00	$ 79,500
Inlet Chamber	1	@ 5000.00	$ 5,000
Miscellaneous: instrumentation, ventilation, hydro			$ 15,000
Sub-total			346,350
Engineering and contingencies 20%			$ 69,650
TOTAL CAPITAL COST			$416,000
Add for concrete lining, if required			$ 82,000
			$498,000
Present Value Cost for Operation and Maintenance			$ 92,000
GRAND TOTAL			$580,000

TABLE 1-9

28,000 m^3 DETENTION TANK, WEST OF ROCKCLIFFE BOULEVARD (PLATE 1-8)

Excavation and backfill	5,100 m^3	@ 3.00	$ 15,300
Excavation and disposal	34,200 m^3	@ 5.50	$ 188,100
Reinforced concrete in forms	2,900 m^3	@ 350.00	$1,015,000
Reinforced concrete on grade	2,280 m^3	@ 200.00	$ 456,000
1200 mm dia. sewer east side	34 m	@ 670.00	$ 22,780
Manholes	2	@ 7000.00	$ 14,000
450 mm dia.sewer south side	53 m	@ 1500.00	$ 79,500
Manholes on Metro. trunk	2	@ 8000.00	$ 16,000
1050 mm dia.sewer from Site 3 overflow	160 m	@ 330.00	$ 52,800
1200 mm dia.sewer from Site 3 overflow	45 m	@ 400.00	$ 18,000
3 - 900 mm dia.sewers across Black Creek	40 m	@ 2000.00	$ 80,000
100 mm dia. drain	20 m	@ 100.00	$ 2,000
Manhole on Rockcliffe culvert	1	@ 10000.00	$ 10,000
Manhole on culvert from Site 3	1	@ 10000.00	$ 10,000
Manholes on sewer from Site 3	2	@ 8000.00	$ 16,000
Restoration:	18,000 m^2	@ 3.00	$ 54,000
Miscellaneous: access road, entrance building, stairways, instrumentation, ventilation, hydro and water supply etc.			$ 30,000
Sub-total			$2,079,480
Engineering and contingencies 20%			$ 415,520
TOTAL CAPITAL COST			$2,495,000
Present Value Cost for Operation and Maintenance			$ 188,000
GRAND TOTAL			$2,683,000

TABLE 1-10

28,000 m^3 DETENTION BASIN, WEST OF ROCKCLIFFE BOULEVARD (PLATE 1-9)

Topsoil stripping & stockpile	4,000 m^3	@ 1.75	$ 7,000
Excavation and disposal	8,200 m^3	@ 5.50	$451,000
Underdrains			$ 13,000
Sodding	27,000 m^2	@ 3.00	$ 81,000
Fencing	500 m	@ 28.00	$ 14,000
Inlets	3	@ 12000.00	$ 36,000
Outlet	1	@ 1200.00	$ 1,200
450 mm dia.sewer to Metro trunk	70 m	@ 1200.00	$ 84,000
Manholes on Metro. trunk	2	@ 8000.00	$ 16,000
1050 mm dia.sewer from Site 3 overflow	160 m	@ 330.00	$ 52,800
1200 mm dia.sewer from Site 3 overflow	45 m	@ 400.00	$ 18,000
3 - 900 mm dia.sewers across Black Creek	40 m	@ 2000.00	$ 80,000
100 mm dia. drain	20 m	@ 100.00	$ 2,000
Manhole on Rockcliffe culvert	1	@ 10000.00	$ 10,000
Manhole on culvert from Site 3	1	@ 10000.00	$ 10,000
Manholes on sewer from Site 3	2	@ 8000.00	$ 16,000
Miscellaneous: instrumentation, ventilation, hydro water supply etc.			$ 15,000
Sub-total			907,000
Engineering and contingencies 20%			$ 181,000
TOTAL CAPITAL COST			$1,088,000
Add for concrete lining, if required			$ 135,000
			$1,223,000
Present Value Cost for Operation and Maintenance			$ 109,000
GRAND TOTAL			$1,332,000

TABLE 1-11

4,200 m^3 DETENTION TANK, EAST OF STEPHEN DRIVE (PLATE 1-10)

Excavation and backfill	1,700 m^3 @ 3.00	$	5,100
Excavation and disposal	4,800 m^3 @ 5.50	$	26,400
Reinforced concrete in forms	800 m^3 @ 350.00	$	280,000
Reinforced concrete on grade	590 m^3 @ 200.00	$	118,000
Overflow structure on existing trunk and connections to trunk		$	15,000
Re-locate existing 675 mm storm sewer	84 m @ 260.00	$	21,840
Manholes	3 @ 2500.00	$	7,500
Headwall		$	5,000
Restoration	3,200 m^2 @ 3.00	$	9,600
Miscellaneous: access road, entrance building, stairways, instrumentation, ventilation, hydro, water supply etc.		$	30,000
Sub-total		$	518,440
Engineering and contingencies 20%		$	103,560
TOTAL CAPITAL COST		$	621,000
Present Value Cost for Operation and Maintenance		$	98,000
GRAND TOTAL		$	719,000

TABLE 1-12

4,200 m^3 DETENTION BASIN AT BERRY ROAD OVERFLOW (PLATE 1-11)

Cost estimate based on (1) obtaining fill from borrow on site, and (2) work carried out during summer months.

Topsoil stripping on borrow area	10,200m^3	@ 2.00	$ 20,400
Excavate muck and top soil in basin area	7,100 m^3	@ 2.50	17,750
Place suitable fill for pond	13,300 m^3	@ 3.00	$ 39,900
Replace topsoil at basin and borrow area	31,800 m^2	@ 1.00	$ 31,800
Seed borrow area and areas disturbed during construction	23,000 m^2	@ 0.50	$ 11,500
Sodding of basin	9,300 m^2	@ 2.00	$ 18,600
Fencing	200 m	@ 28.00	$ 5,600
900 mm dia.sewer	154 m	@ 525.00	$ 81,000
2400 mm dia. sewer	30 m	@ 1500.00	$ 45,000
100 mm dia. forcemain	172 m	@ 40.00	$ 6,880
Manholes	3	@ 4000.00	$ 12,000
Headwalls	1	@ 9000.00	$ 9,000
	1	@ 17000.00	$ 17,000
Pumping manhole with dual submersible pumps			$ 50,000
Restoration			$ 7,000
Miscellaneous: access road, instrumentation hydro, etc.			$ 54,000
Sub-total			$427,430
Engineering and contingencies 20%			$ 85,570
TOTAL CAPITAL COST			$513,000
Add for concrete lining, if required			$ 41,000
			$554,000
Present Value Cost for Operation and Maintenance			$ 84,000
GRAND TOTAL			$638,000

TABLE 1-13

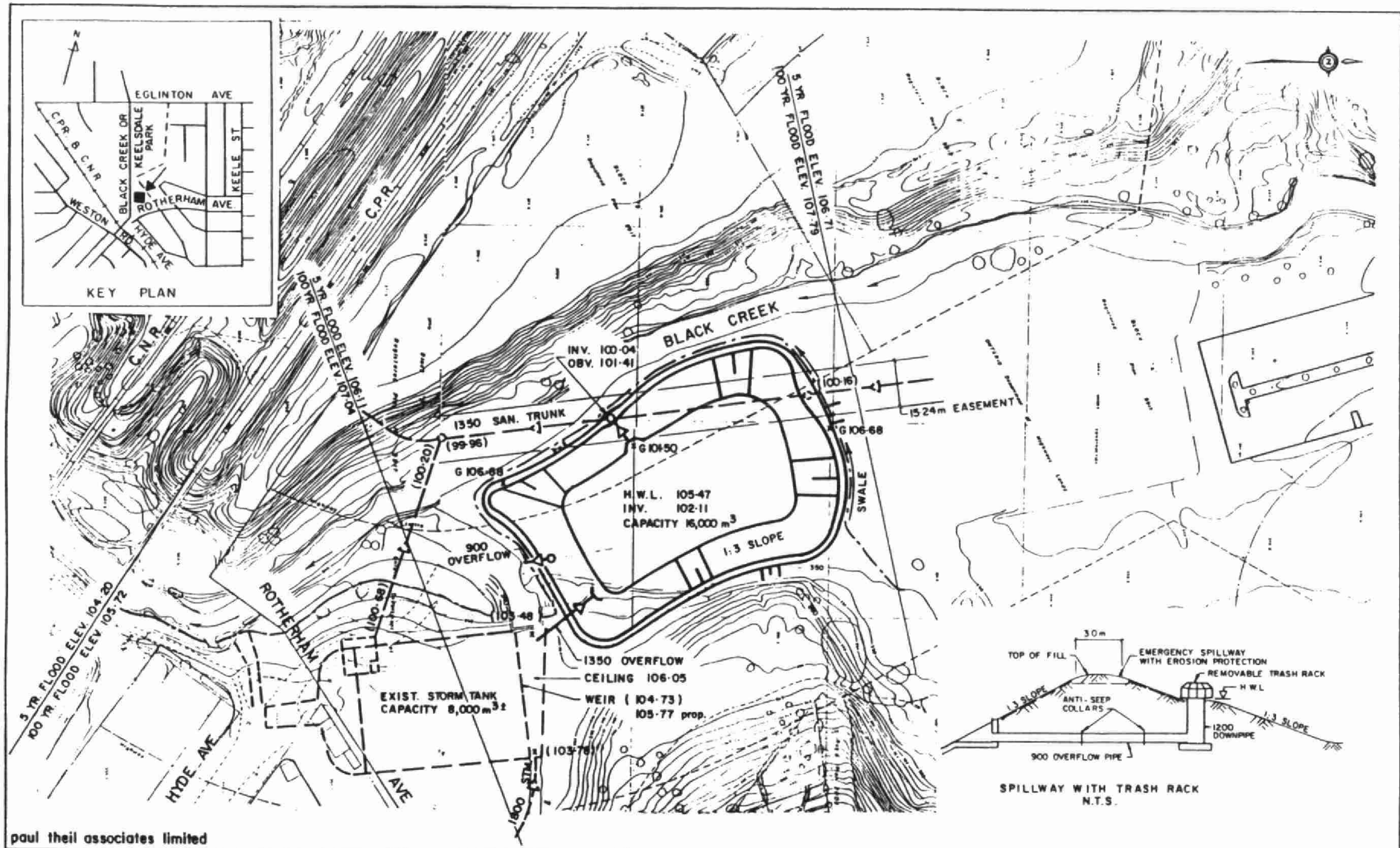
4,200 m^3 DETENTION BASIN AT HUMBER W.P.C.P. (PLATE 1-12)

Topsoil stripping & stockpile	1,500 m^3 @ 2.00	$ 3,000
Excavation and disposal	8,200 m^3 @ 5.50	$ 45,100
Underdrains		$ 2,000
Sodding	4,900 m^2 @ 3.00	$ 14,700
Fencing		$ 6,000
Overflow chamber on existing 2700 mm dia. trunk sanitary		$ 15,000
Inlet-Outlet structures		$ 15,000
900 mm dia.sewer from trunk san.	70 m @ 300.00	$ 21,000
450 mm dia. outlet to existing building	85 m @ 180.00	$ 15,300
Manhole for 450 mm dia. sewer including submersible pumps		$ 25,000
Retaining wall		$ 55,000
Concrete invert		$ 20,000
Road restoration		$ 5,000
Miscellaneous: instrumentation, hydro,water supply etc.		$ 15,000
Sub-total		$257,100
Engineering and contingencies 20%		$ 51,900
TOTAL CAPITAL COST		$309,000
Present Value Cost for Operation and Maintenance		$ 89,000
GRAND TOTAL		$393,000

TABLE 1-14

SANITARY RELIEF SEWER FROM SITE 3 OVERFLOW TO HUMBER SANITARY TRUNK SEWER (PLATES 1-13 AND 1-14)

900 mm dia.pipe Class III	425 m	@ 129.00	$ 54,825
" " " Class IV	1,530 m	@ 146.00	$223,380
" " " Class V	218 m	@ 170.00	$ 37,060
Bedding: 0.99 m^3/m =	2,151 m^3	@ 20.00	$ 43,020
Manholes:	13	@ 3800.00	$ 49,400
Installation: (assume wet ground requiring closed sheeting)			
Timber (recycled)	2,200 m	@ 5.00	$ 11,000
Daily Costs: $4,500			
Aver.daily production: 15 m			
Labour costs: $\frac{2173}{15}$ x 4500			$652,000
Removal of surplus material:			
1.4 m^3/m =	3,000 m^3	@ 5.50	$ 16,500
Restoration			$125,000
Sub-total			$1,212,185
Engineering and contingencies 20%			$ 242,815
TOTAL CAPITAL COST			$1,455,000



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

DETENTION BASIN AT HYDE AVENUE

Serving: Hillary Drainage Area

0 10 20 30 40 50
scale metres

legend

525	existing sewer (mm)
450	proposed sewer (mm)
(100·50)	existing invert elevation (m)
101·00	proposed invert elevation (m)
G 104·50	proposed ground elevation (m)
— · — · —	5 yr. Flood Line with works in place

Note: contours are in feet

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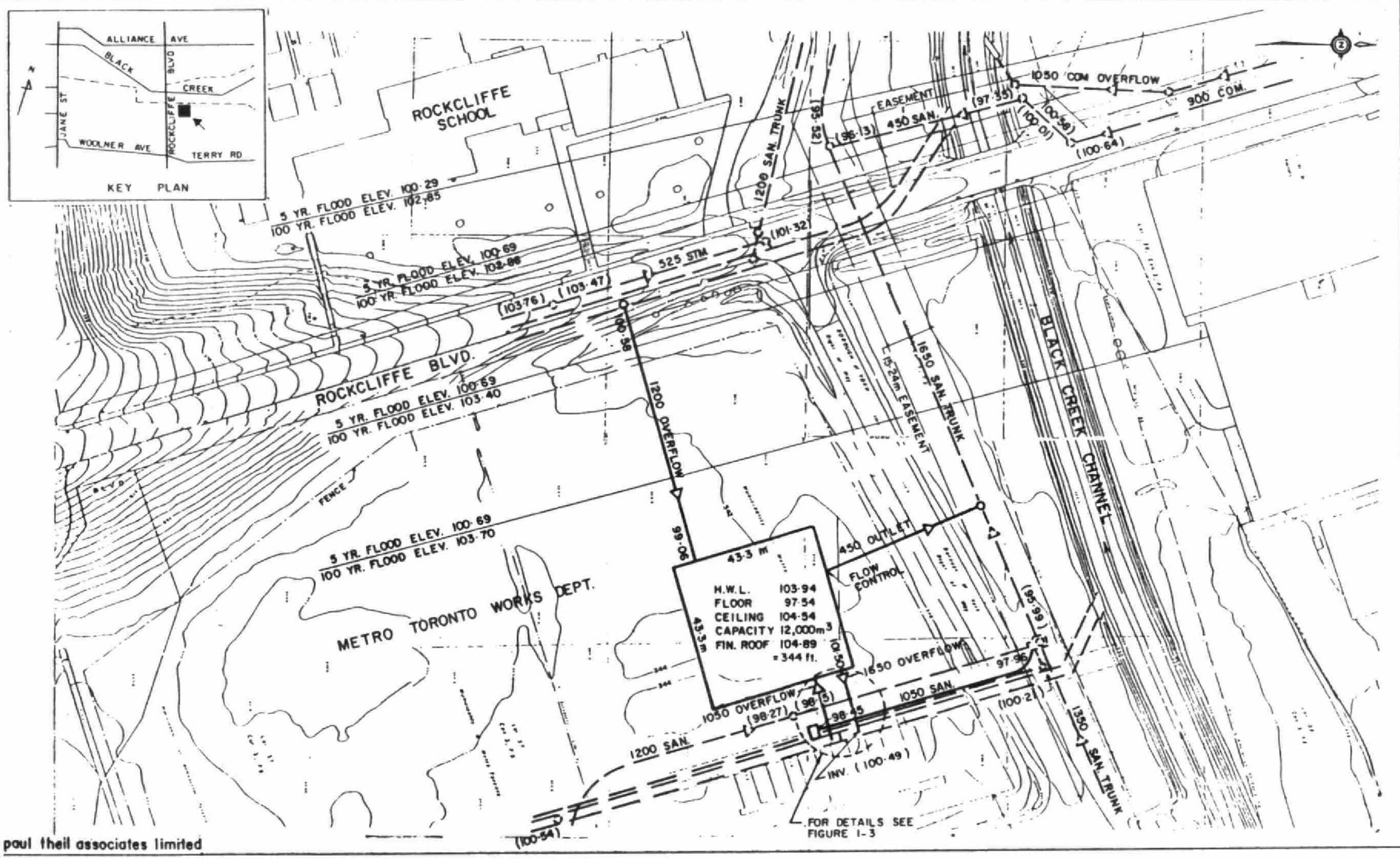
plate I-1



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Feasibility Study And Costing Of
Proposed Pollution Control Measures In The Humber Sewershed

DETENTION TANK EAST OF ROCKCLIFFE BOULEVARD

Serving: Rockcliffe Area Site 3 Regulator

scale 0 10 20 30 40 50 metres

legend

525	existing sewer (mm)
450	proposed sewer (mm)
(100·50)	existing invert elevation (m)
101·00	proposed invert elevation (m)
G 104·50	proposed ground elevation (m)

Note:
1 contours are in feet
2 5 yr. floodlines are below the top of banks of Black Creek channel

plate 1-2

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Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

DETENTION BASIN EAST OF ROCKCLIFFE BOULEVARD

Serving:
Rockcliffe Area Site 3 Regulator

0 10 20 30 40 50
scale metres

legend

525 existing sewer (mm)

450 proposed sewer (mm)

(100·50) existing invert elevation (m)

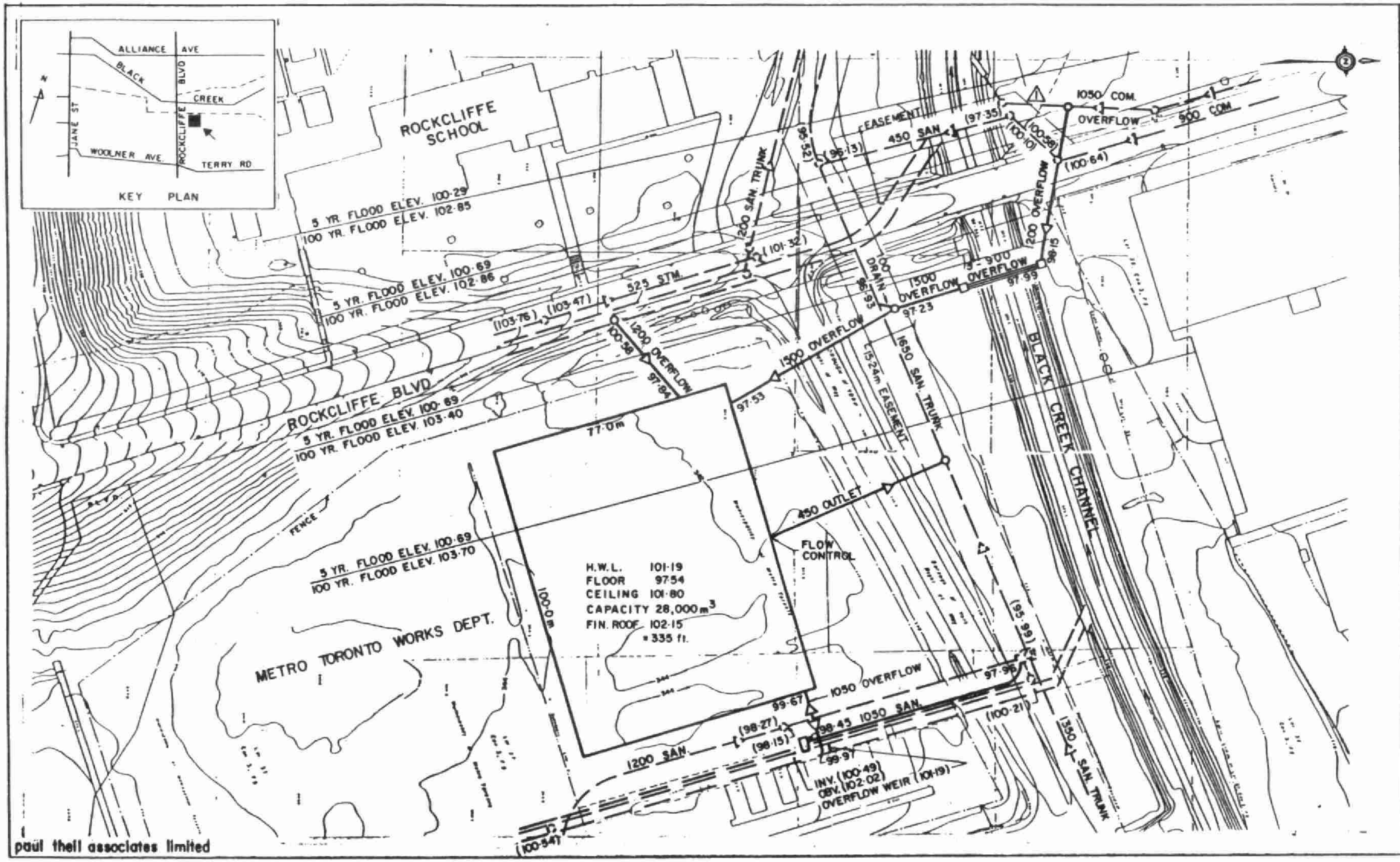
101·00 proposed invert elevation (m)

G 104·50 proposed ground elevation (m)

Note:

1. contours are in feet
2. 5 yr. floodlines are below the top of banks of Black Creek channel

plate 1-3



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Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

DETENTION TANK EAST OF ROCKCLIFFE BOULEVARD

Serving:
Mt. Dennis Area
Rockcliffe Area
Site 3 Regulator

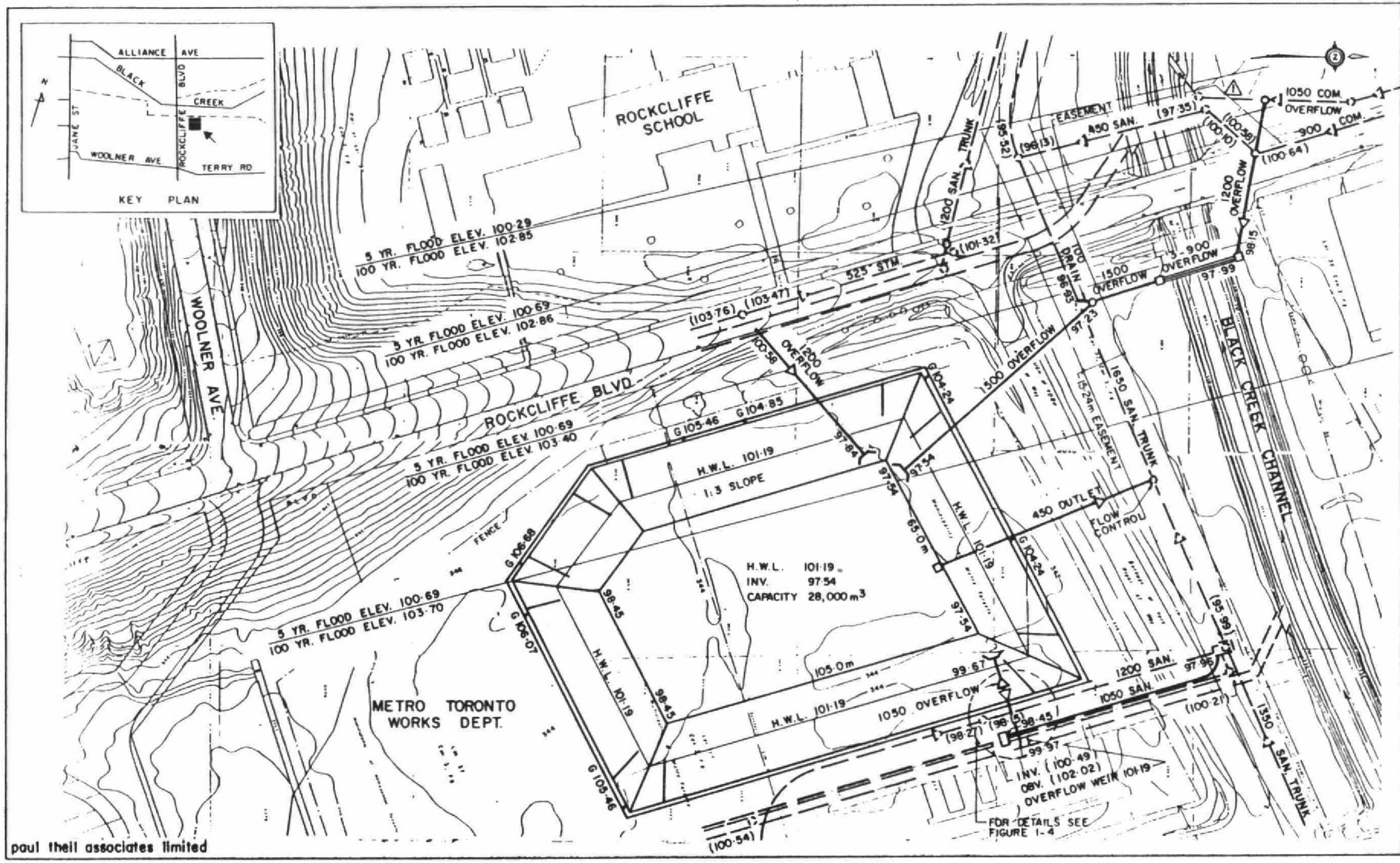
0 10 20 30 40 50
scale metres

legend

525	existing sewer (mm)
450	proposed sewer (mm)
(100·50)	existing invert elevation (m)
101·00	proposed invert elevation (m)
G 104·50	proposed invert elevation (m)

Note:
1. contours are in feet
2. 5 yr. floodlines are below the top of banks of Black Creek channel

plate I-4



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

DETENTION BASIN EAST OF ROCKCLIFFE BOULEVARD

Serving:
Mt. Dennis Area
Rockcliffe Area
Site 3 Regulator

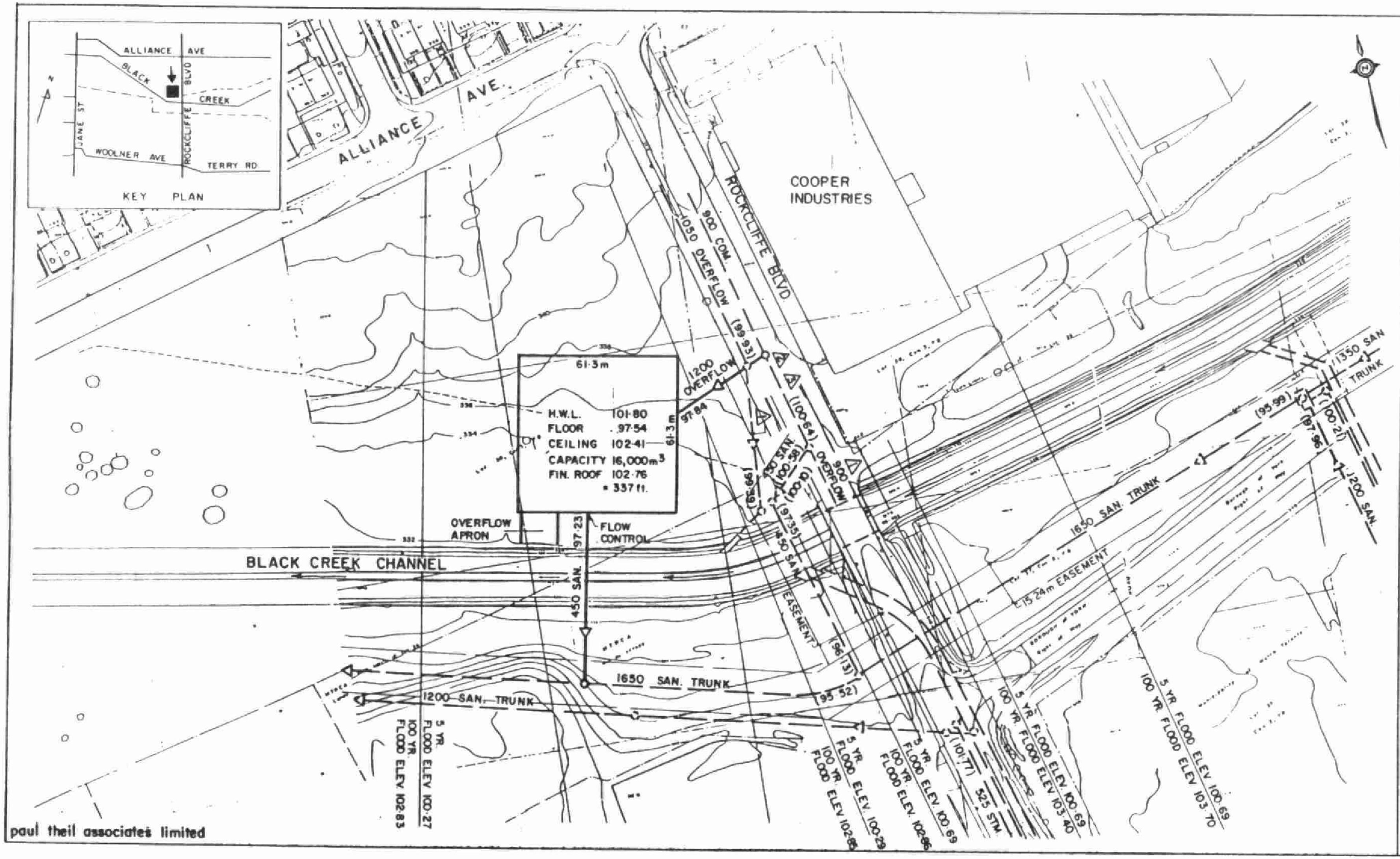
0 10 20 30 40 50
scale metres

legend

525	existing sewer (mm)
450	proposed sewer (mm)
(100·50)	existing invert elevation (m)
101·00	proposed invert elevation (m)
G 104·50	proposed ground elevation (m)
△	to be abandoned

Note·
1 contours are in feet
2 5 yr. floodlines are below the top of banks of Black Creek channel

plate 1-5



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

DETENTION TANK WEST OF ROCKCLIFFE BOULEVARD

Serving: Mt. Dennis Area

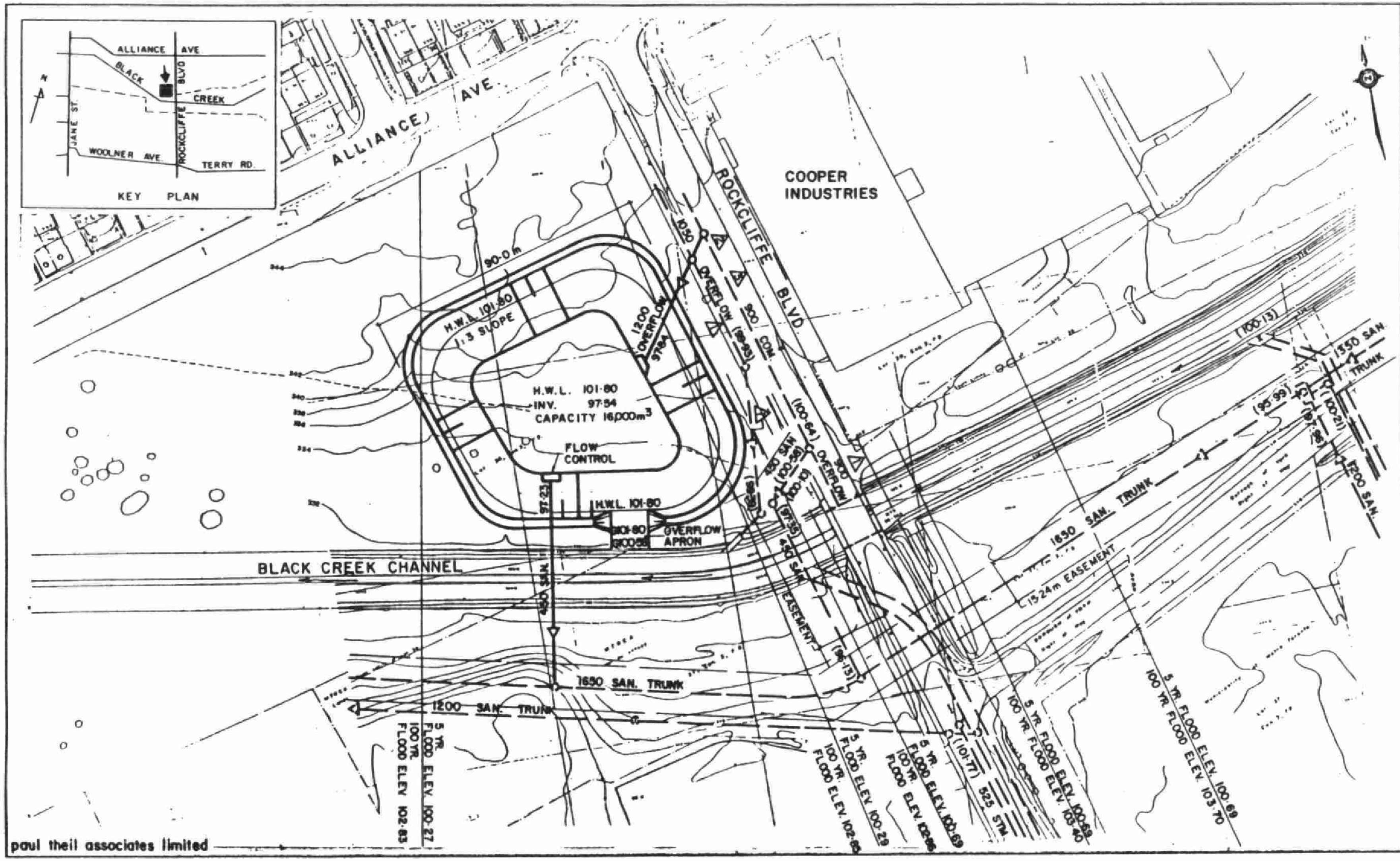
scale 0 10 20 30 40 50 metres

legend

- 525 — existing sewer (mm)
- 450 — proposed sewer (mm)
- (100·50) existing invert elevation (m)
- 101·00 proposed invert elevation (m)
- G 104·50 proposed ground elevation (m)
- ⚠1 to be abandoned
- ⚠2 new diversion chamber
- ⚠3 converts to sanitary sewer

Note:
1 contours are in feet
2 5 yr floodlines are below the top of banks of Black Creek channel

plate 1-6



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

DETENTION BASIN WEST OF ROCKCLIFFE BOULEVARD

Serving: Mt. Dennis Area

0 10 20 30 40 50
scale metres

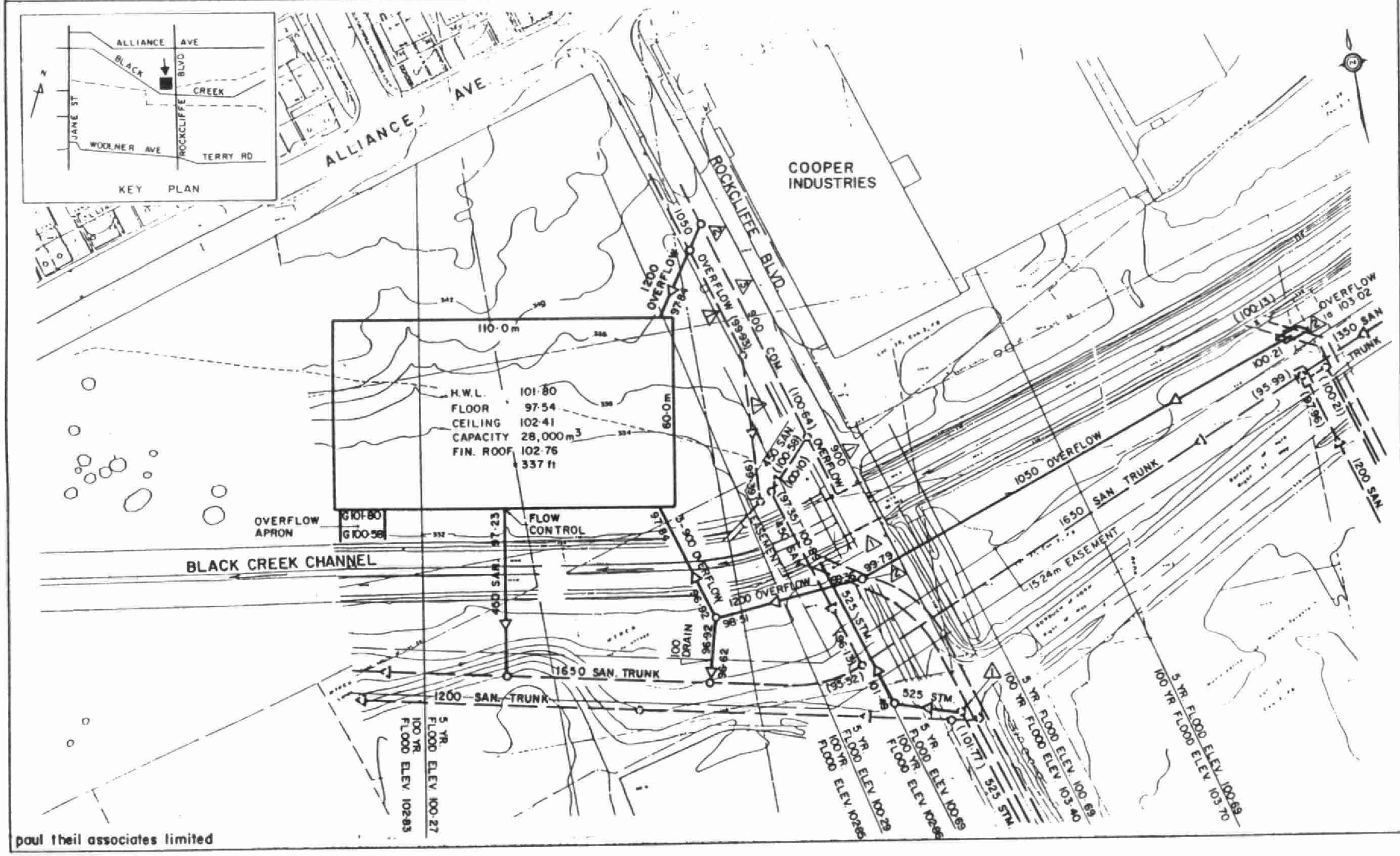
legend

- 525 existing sewer (mm)
- 450 proposed sewer (mm)
- (100·50) existing invert elevation (m)
- 101·00 proposed invert elevation (m)
- G 104·50 proposed ground elevation (m)
- 1 to be abandoned
- 2 new diversion chamber
- 3 converts to sanitary sewer

Note

1. contours are in feet
2. 5 yr. floodlines are below the top of banks of Black Creek channel

plate 1-7



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

DETENTION TANK WEST OF ROCKCLIFFE BOULEVARD

Serving:
Mt. Dennis Area
Rockcliffe Area
Site 3 Regulator

scale 0 10 20 30 40 50 metres

legend

- 525 existing sewer (mm)
- 450 proposed sewer (mm)
- (100·50) existing invert elevation (m)
- 101·00 proposed invert elevation (m)
- G 104·50 proposed ground elevation (m)
- 1 to be abandoned
- 2 new diversion chamber
- 3 converts to sanitary sewer

Note:
1 contours are in feet
2 5 yr. floodlines are below the top of banks of Black Creek channel

plate I-8

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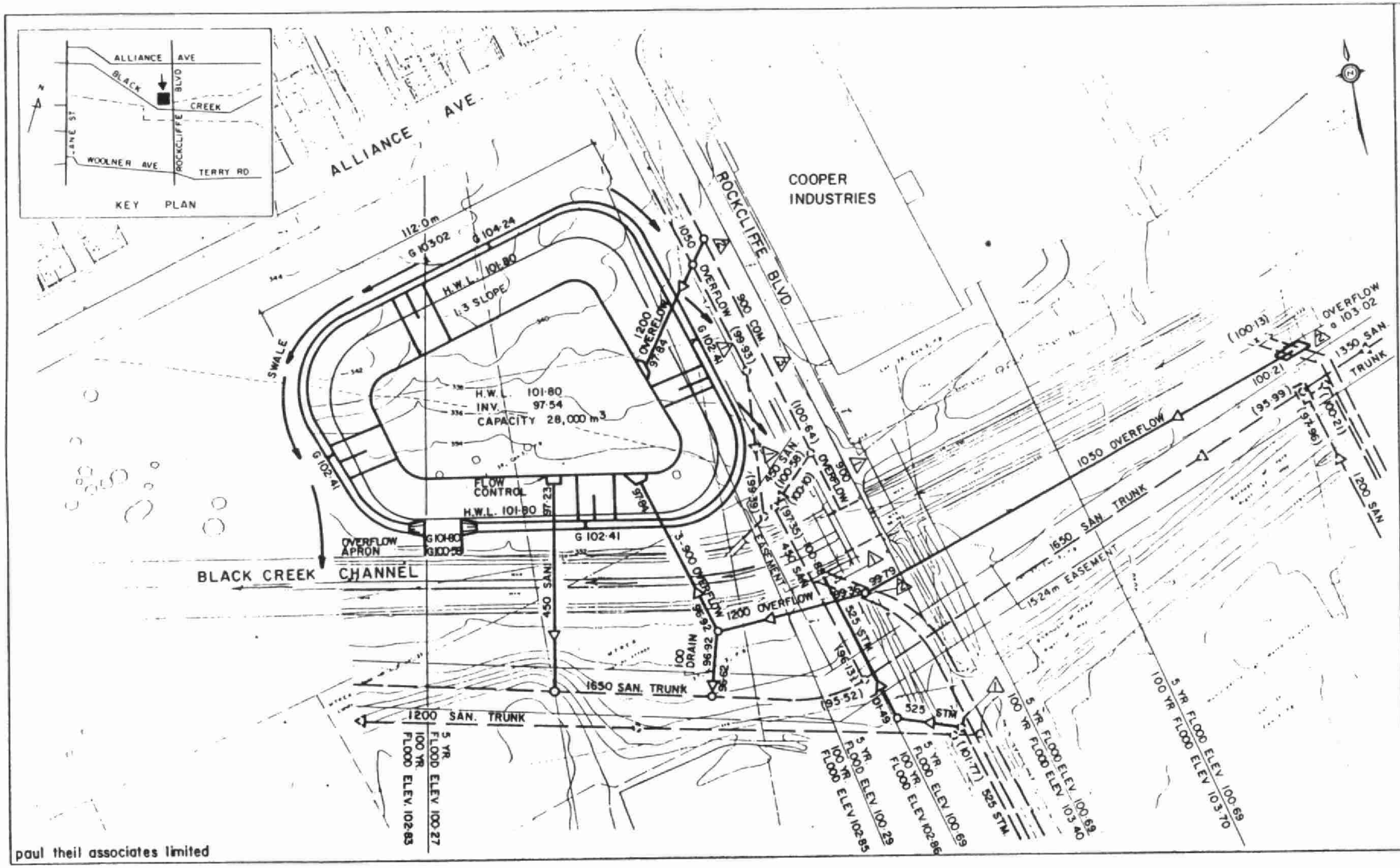

Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

DETENTION BASIN WEST OF ROCKCLIFFE BOULEVARD

Serving:

Mt. Dennis Area
Rockcliffe Area
Site 3 Regulator

scale 0 10 20 30 40 50 metres

legend

525	existing sewer (mm)
450	proposed sewer (mm)
(100.50)	existing invert elevation (m)
101.00	proposed invert elevation (m)
G 104.50	proposed ground elevation (m)
1	to be abandoned
2	new diversion chamber
3	converts to sanitary sewer

Note:

1. contours are in feet
2. 5 yr. floodlines are below the top of banks of Black Creek channel

plate 1-9

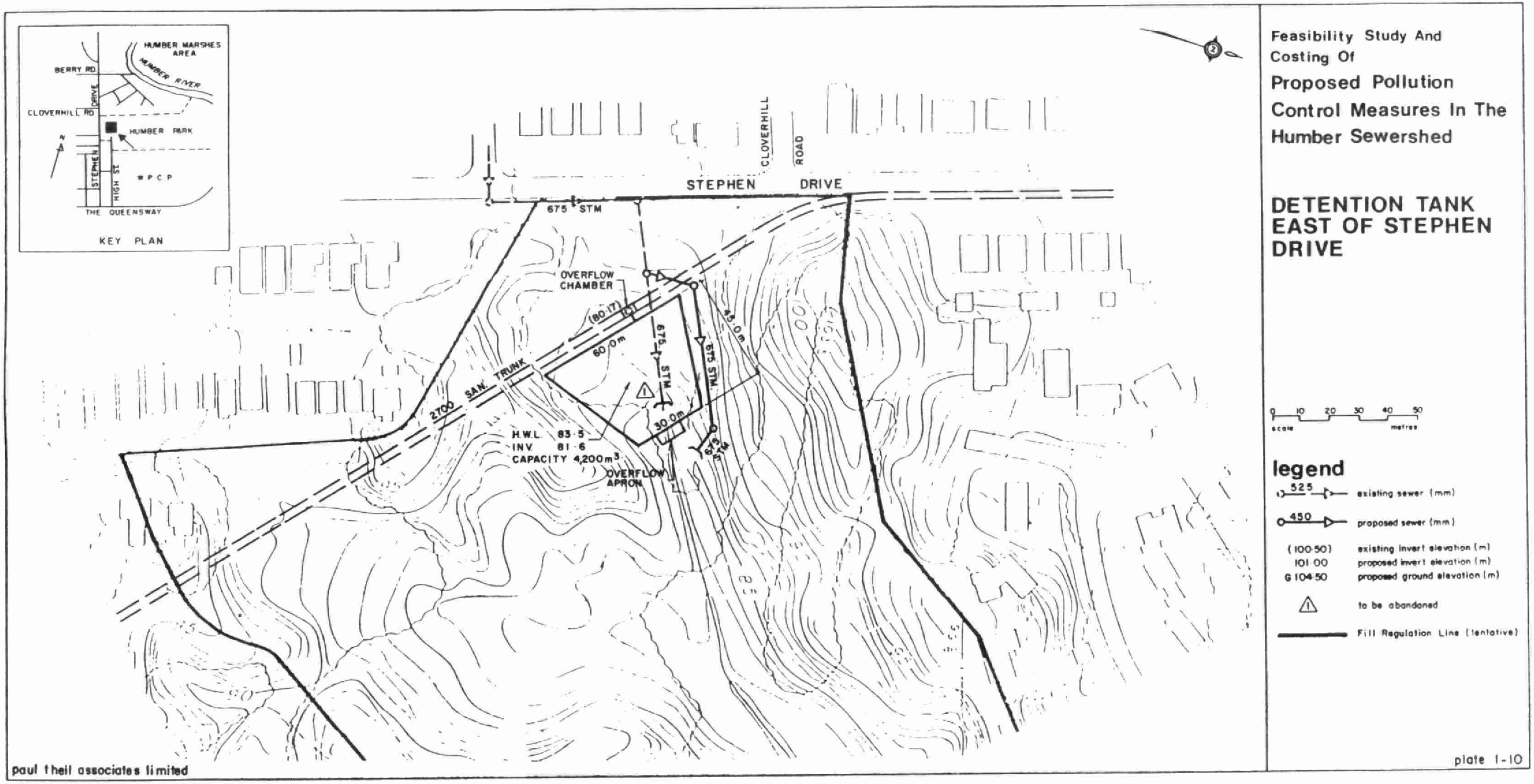
HUMBER MARSHES AREA
BERRY RD.
HUMBER RIVER
DRIVE
CLOVERHILL RD
HUMBER PARK
N
STEPHEN
HIGH ST
W.P.C.P.
THE QUEENSWAY
KEY PLAN
CLOVERHILL
ROAD
STEPHEN DRIVE
675 STM
OVERFLOW CHAMBER
(80.17)
45.0m
675 STM
675 STM.
60.0m
2700 SAN. TRUNK
30.0m
675 STM
H.W.L. 83.5
INV. 81.6
CAPACITY 4,200m³
OVERFLOW APRON
Feasibility Study And Costing Of
Proposed Pollution Control Measures In The Humber Sewershed
DETENTION TANK EAST OF STEPHEN DRIVE
0 10 20 30 40 50
scale metres
legend
525 existing sewer (mm)
450 proposed sewer (mm)
(100.50) existing invert elevation (m)
101.00 proposed invert elevation (m)
G 104.50 proposed ground elevation (m)
to be abandoned
Fill Regulation Line (tentative)
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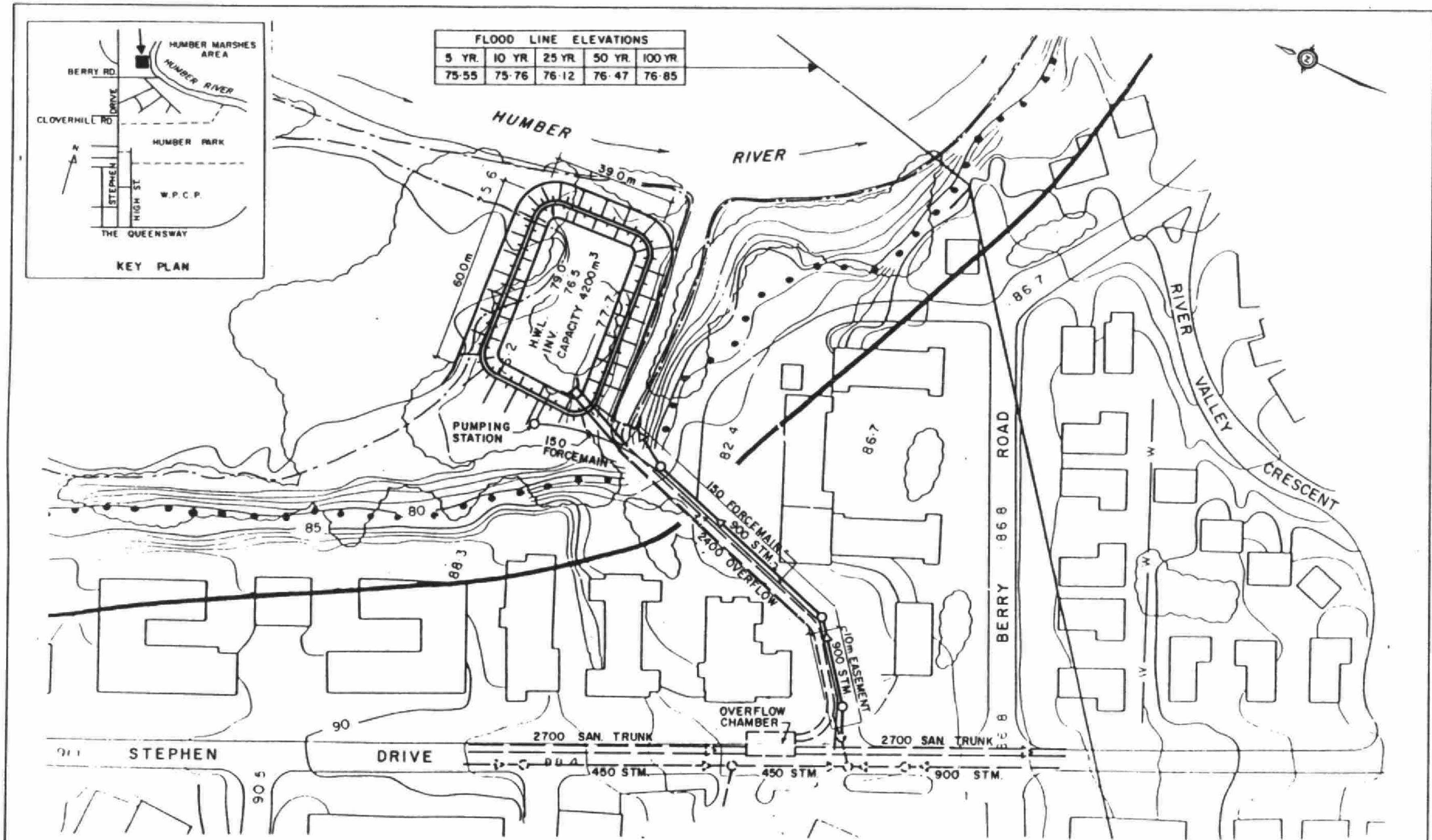
plate 1-10



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Feasibility Study And Costing Of

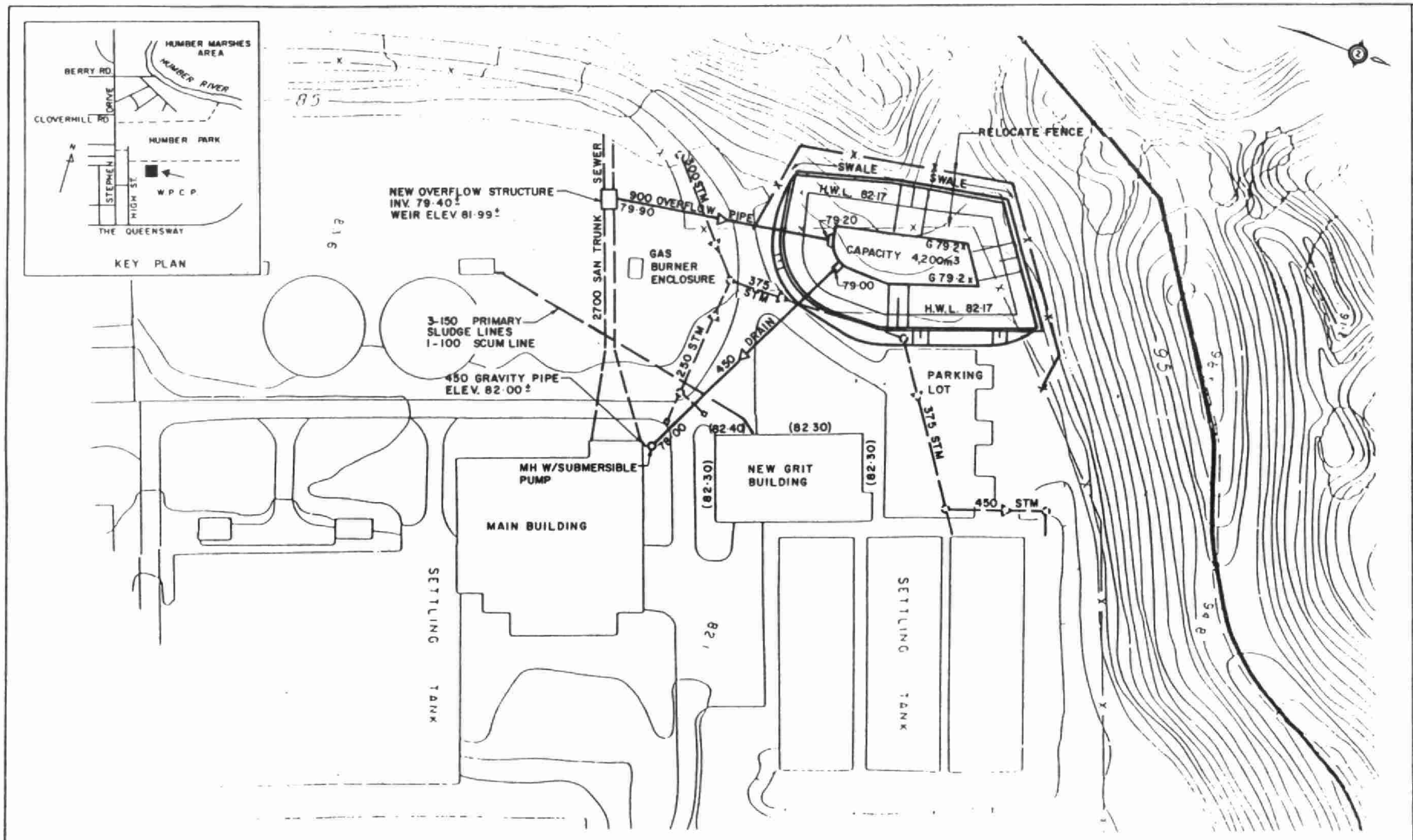
Proposed Pollution Control Measures In The Humber Sewershed

DETENTION BASIN AT BERRY ROAD OVERFLOW

scale 0 10 20 30 40 50 metres

legend

- 450 existing sewer (mm)
- 900 proposed sewer (mm)
- • • • Regional Flood Line (tentative)
- 5 yr. Flood Line
- Fill Regulation Line (tentative)



Feasibility Study And Costing Of

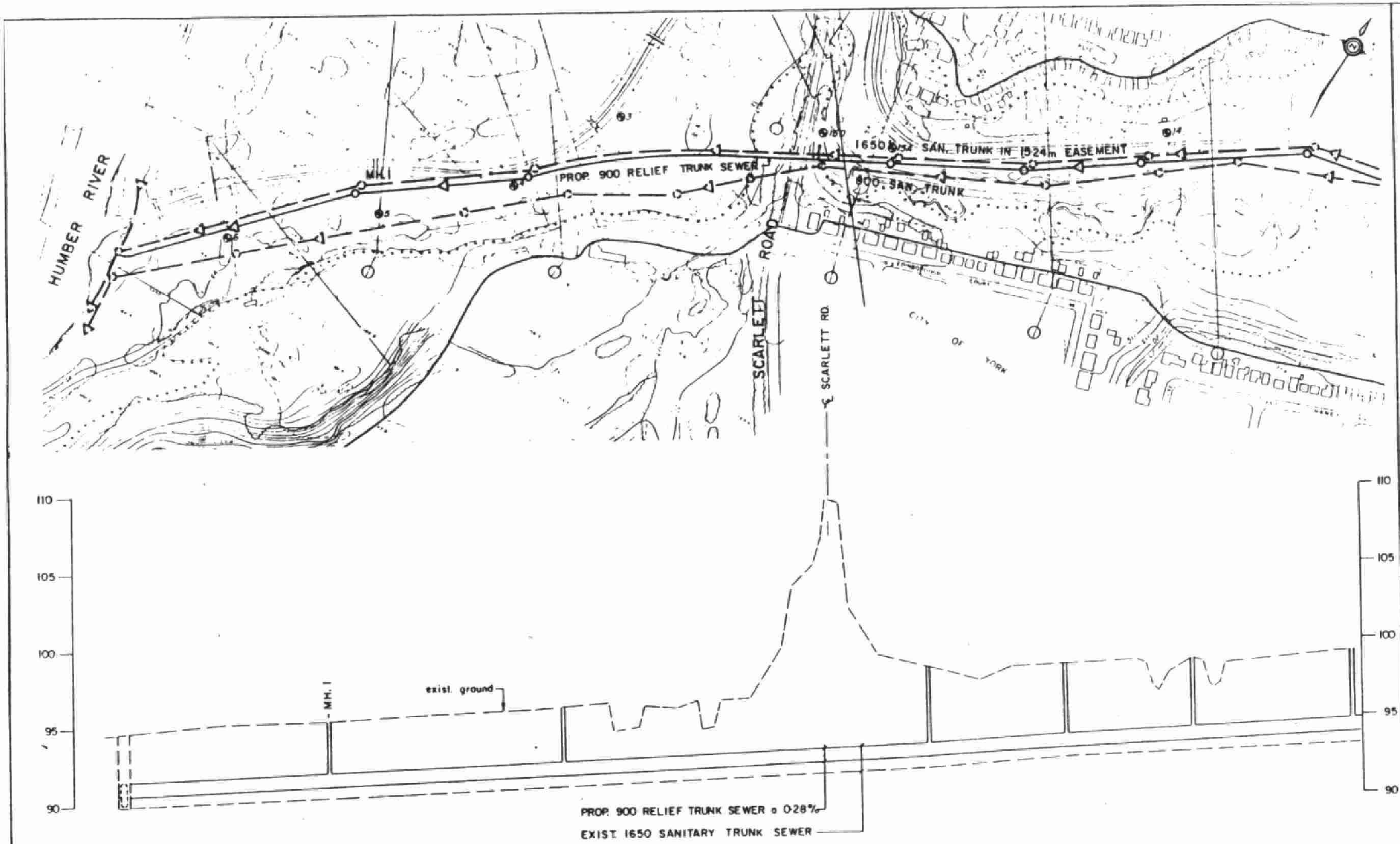
Proposed Pollution Control Measures In The Humber Sewershed

DETENTION BASIN AT HUMBER W.P.C.P.

scale 0 10 20 30 40 50 metres

legend

525	existing sewer (mm)
450	proposed sewer (mm)
(100·50)	existing invert elevation (m)
101·00	proposed invert elevation (m)
G 104·50	proposed ground elevation (m)
	Fill Regulation Line (tentative)



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

BLACK CREEK SANITARY RELIEF SEWER (Part 1)

SCALE - horz. 1:3,000
vert. 1:200

legend

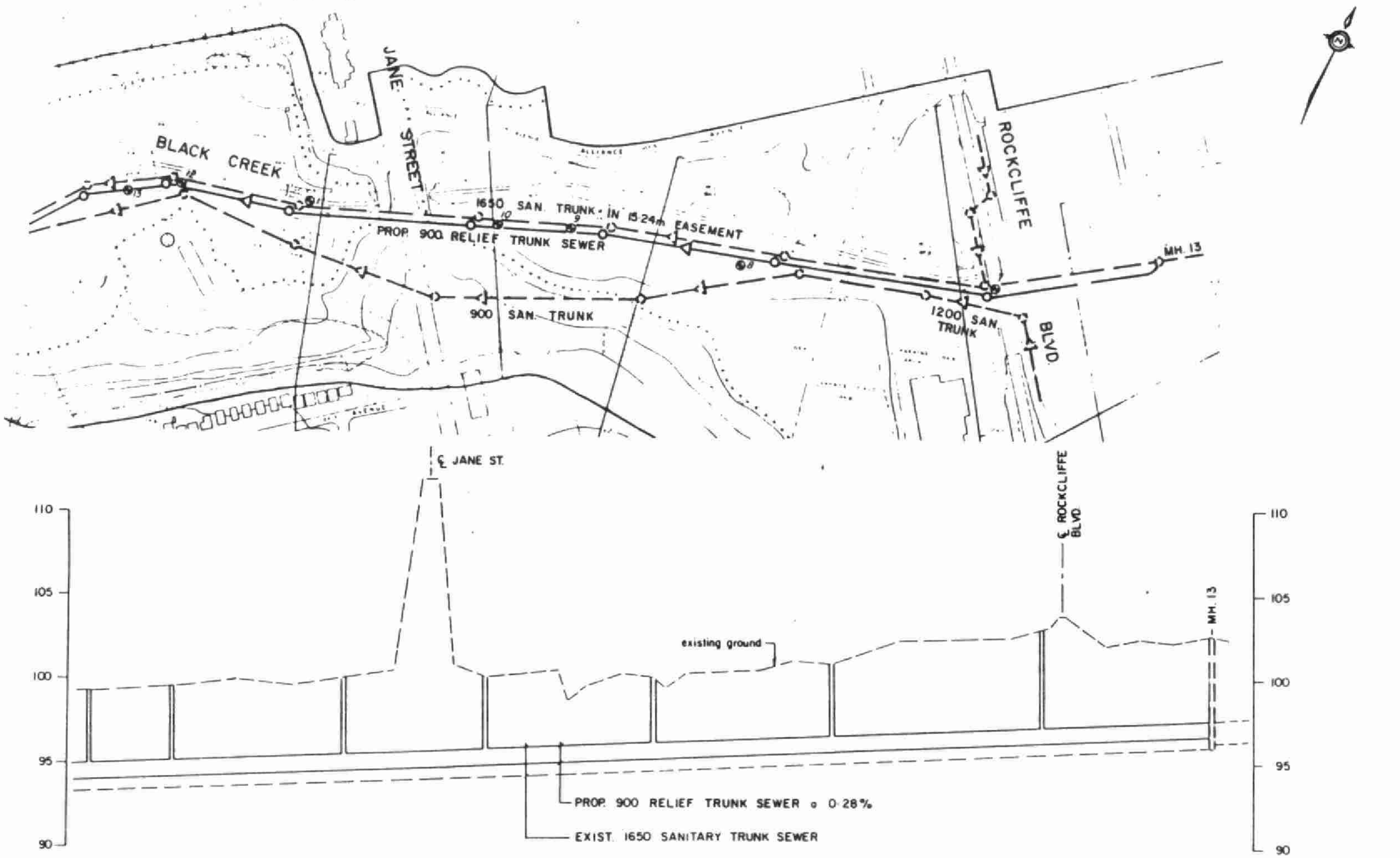
525 existing sewer (mm)

450 proposed sewer (mm)

borehole location and number

plate 1-13

paul theil associates limited



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

BLACK CREEK SANITARY RELIEF SEWER (Part 2)

SCALE - horz 1: 3,000
vert 1: 200

legend

525 existing sewer (mm)

450 proposed sewer (mm)

10 borehole location and number

paul theil associates limited

SECTION 2

TASK 2

Flow Control in Local Combined Sewers

TASK 2

FLOW CONTROL IN LOCAL COMBINED SEWERS

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FEASIBILITY STUDY AND COSTING OF
PROPOSED POLLUTION CONTROL MEASURES
IN THE HUMBER SEWERSHED

TASK 2

FLOW CONTROL IN LOCAL COMBINED SEWERS

1 - INTRODUCTION

1.1 General

The City of York has, in recent years, been implementing a scheme which will separate the original combined sewer system. There remains, however, a large percentage of the City which is still serviced by combined sewers. These sewers, some of which were constructed 60 years ago, have insufficient capacity to handle the storm water runoff from the more intense storms. This has resulted in numerous occurrences of basement floodings within the City. In addition, the design of the existing combined sewer system results in frequent combined sewer overflows to Black Creek each year.

1.2 Objectives

The objectives of Task 2 are to analyse the existing combined sewer systems within the City in order to establish the remedial works which will eliminate basement flooding under selected design storm events, and reduce the frequency and magnitude of combined sewer overflows to the Black Creek. The types of remedial works which will be considered in this Task are primarily inlet control and detention tanks. The results from this Task will enable the management of the TAWMS to compare the merits of this approach with those of other methods which may be used to achieve similar objectives.

1.3 Study Methodology

Our approach to this Task was to select one area within the City of York and analyse this area in detail. The results from the area studied in detail would then be used as a basis to estimate the remedial works required and to develop estimated costs for other areas within the City. Ultimately the total estimated costs to meet the above two objectives would be established.

The combined sewer system which was selected for detailed analysis is referred to as the Hillary Avenue sewer system (Plate 2-1). The drainage area of this sewer system is about 630 ha which represents roughly 75 percent of the total area within the City which is still serviced by combined sewers.

The initial step of the study was a thorough review of all relevant data pertaining to recorded combined sewer overflow and flooding events. Plan and profiles of the existing Hillary Avenue combined sewer system were obtained from the City of York Works Department. Inconsistencies, questionable data or missing data were verified by City staff. Drainage limits for the combined sewer system were determined, the primary hydraulic elements affecting combined sewer overflows and basement flooding were identified, and a representative computer model of the Hillary Avenue combined sewer system was developed. Site visits were conducted in order to become familiar with the physical characteristics of the area.

A series of simulations to calibrate the computer model to measured records obtained for a combined sewer overflow event and for a flooding event was undertaken. The primary purpose of the calibration procedure is to set model runoff parameters which dictate the hydraulic response of the computer model. The objective is to establish runoff parameters which produce, at the planning or feasibility stage, results consistent with measured records. The calibration procedure should ensure that reasonable results for any subsequent storm which is modelled will be attained.

Satisfied that the model reasonably represented the combined sewer system, the existing system was simulated for 2, 5 and 10 year design storm events and for five rainfall events during the April to October 1979 period. The results from the simulations illustrated the magnitude and location of the deficiences within the sewer system for the design storms and the magnitude of the combined sewer overflows for the 1979 storm events.

Following the simulation of the design storms, numerous alternative solutions were generated. The objective of each alternative was to eliminate basement flooding within the Hillary Avenue system and with these works in place, establish what additional works would be required to eliminate combined sewer overflows for each of the five storm events in 1979. A cost estimate was then prepared for each practical alternative. Several estimates have been prepared where alternative methods for installing the remedial works are feasible.

The cost estimates for the Hillary Avenue system were then used to estimate costs for other areas within the City of York which remain serviced by combined sewers. The total costs for various remedial measures are given in section 6.

2 - DESCRIPTION OF THE STUDY AREA

The Hillary Avenue sewer system services an area which is bounded roughly by the Black Creek to the west, the City limits to the north and south, and Bathurst Street to the east (Figure 2-1). The area is a relatively older section of Metropolitan Toronto with some homes being built shortly after the turn of the century. Over the last 20 years redevelopment has taken place which increased the population density and reduced the amount of open space areas.

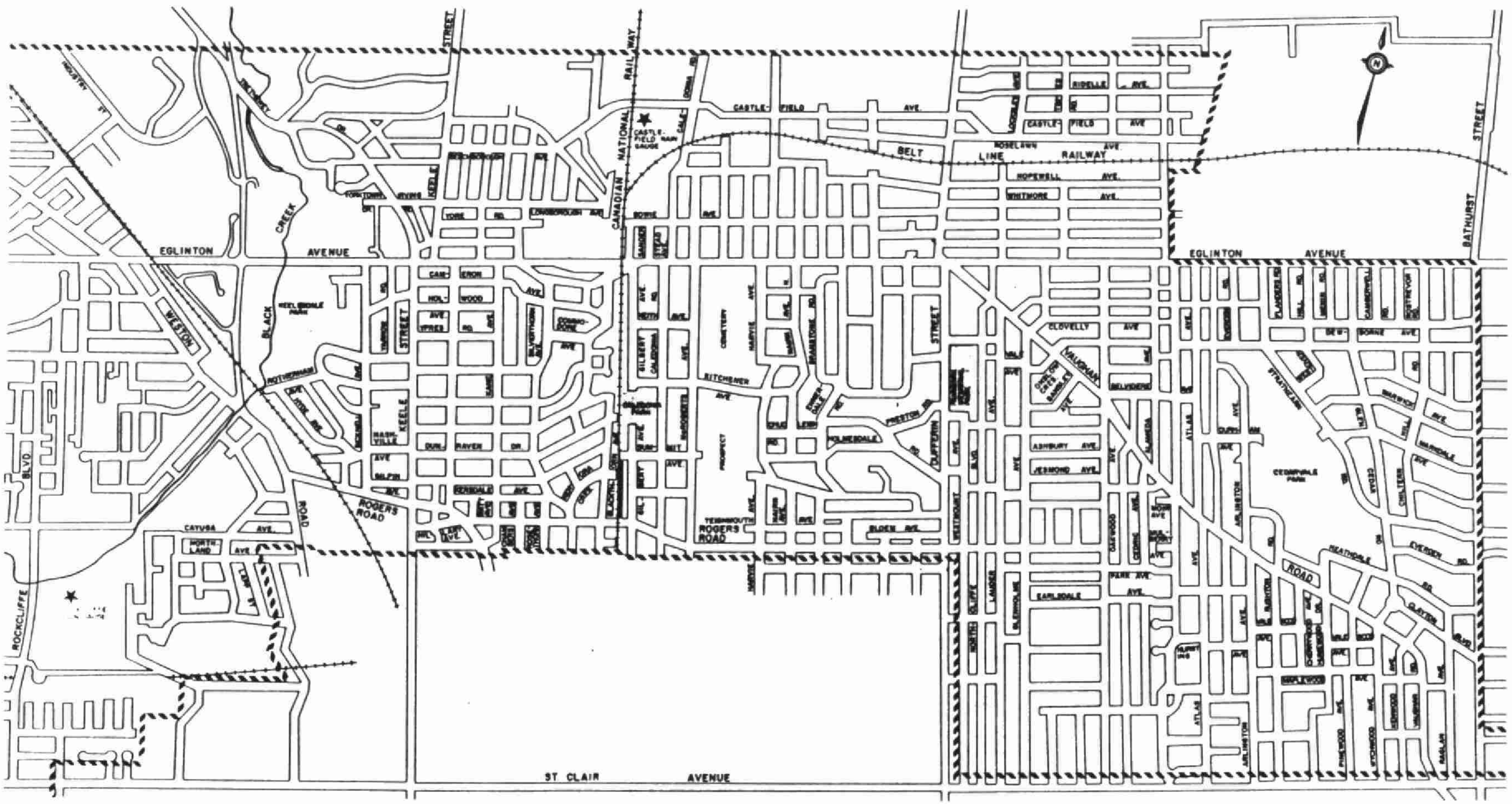
Land use within the area is primarily residential. A major industrial area is located along Castlefield Avenue between Keele Street and Dufferin Street. Commercial areas exist primarily along Eglinton Avenue, with smaller commercial areas located along Dufferin Street and Rogers Road east of Keele Street. Keelesdale Park, Cedarvale Park and Prospect Cemetery comprise the major open space areas. Smaller open space areas are located along Gilbert Avenue (Caledonia Park), Dufferin Street (Fairbank Memorial Park), and within a few of the primary and secondary school areas.

The topography varies considerably within the area. In general, the lands drain in a southwesterly direction from Bathurst Street and Ridelle Avenue to the Black Creek. However, overland drainage routes are very non-continuous due to the undulating nature of the lands. The steepest section of land is located within the area bounded by Keele Street, Summit Avenue, Eglinton Avenue and Vaughan Road. Areas with gently sloping terrain are found south of Rogers Road and north of Eglinton Avenue.

The soils within the area may be categorized as clayey silts to silt tills, and exhibit poor to moderate drainage characteristics (Reference 1).

2.1 Drainage System,

The sewer system which was modelled is a relatively complex system. In



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

STUDY AREA Hillary Avenue Catchment

0 100 300 500 700
scale metres

legend

city limits

paul theil associates limited

the Hyde Avenue storage tank. These flows, along with any flows which are conveyed by the trunk sewer along Dunraven Drive and Nashville Avenue are stored in the 7320 m^3 Hyde Avenue tank during rainfall events. The flows are then released into the Black Creek sanitary trunk sewer via a 300 mm pipe after the storm has subsided. Flows which exceed the capacity of the Hyde Avenue tank are discharged via a 34.0 m long weir into the Black Creek.

In order to reasonably model the areas which are subject to basement flooding, the City of York flooding records dating back to 1976 were inspected. Based on these records, the sewer segmentation as shown on Plate 2-1 was established. In general, all major trunks and the laterals where basement flooding has been recorded have been modelled in detail. For the remaining areas where no basement flooding has been recorded, or where sewer separation has been carried out, a more simplified segmentation of the sewer system was adopted.

The area bounded by Eglinton Avenue, Black Creek, the north city limits and the Canadian National Railway is serviced primarily by a 1200 mm x 1275 mm sewer along Eglinton Avenue to the Black Creek. Only the flows which are conveyed southerly along Keele Street (via a 450 mm pipe) were included in the modelling. This area, which is partially separated, experiences basement flooding only along streets where storm sewer separation has been completed. Combined sewer overflows which may occur from this area (discharging to the Black Creek via the 1200 mm x 1275 mm sewer along Eglinton Avenue) were not considered in this Task.

To properly account for combined sewer overflows, seven overflow locations were included in the model. The location of the overflows are shown on Plate 2-1 and a description of the type of overflow at each location is given in Table 2-1. In total 295 sewer segments were used to model the Hillary Avenue combined trunk sewer system.

TABLE 2-1

LOCATION AND TYPE OF COMBINED SEWER OVERFLOW STRUCTURES

Overflow No.	Location	Type of Overflow	Direction of Low Flows	Direction of Overflows
OF1	Dunraven Dr.- Kane Ave.	Weir and 600 mm low flow pipe	south along Kane Ave.	west along Dunraven Dr.
OF2	Scott Rd.- Dunraven Dr.	675 mm pipe through manhole cut below spring line	south along Scott Rd.	west along Dunraven Dr.
OF3	Keele St.- Dunraven Dr.	975 mm pipe through manhole cut below spring line	south along Keele St.	west along Nashville Ave
OF4	Hillary Ave.- Rogers Rd.	2 - 1200 mm x 1670 mm box culverts	westerly along Hillary Ave.to OF5	west along Hillary and ultimately to Hyde Ave.tank
OF5	Hillary Ave.- Keele St.	gate valve and overflow weir	south along Keele St. to Site 3 regulator	north along Keele St. to Hyde Ave.tank
OF6	Site 3 Regulator	10.7 m horizontal weir	to Black Creek sanitary trunk sewer	to Black Creek
OF7	Hyde Ave. Tank	34.0 m horizontal weir	to Black Creek sanitary trunk sewer	to Black Creek

2.2 Data Collection and Analysis

Data for this study was collected from various sources. As was mentioned previously, flooding records dating back to 1976 were collected from the City of York. The records indicated that the most severe flooding within the City of York occurred during the summer of 1977 when roughly 2,000 to 3,000 homes were flooded. The depths of water in the basements varied from 5 mm to 900 mm. More recently, a lesser number of homes (about 70 to 280) have reported basement flooding problems in the summers of 1982, 1983 and 1984.

Rainfall data was obtained from the Ministry of the Environment (MOE), the Canadian Atmospheric Environment Services (AES), and the City of York. The City of York have a rain gauge which is located in the Works Yard at Castlefield Avenue (Figure 2-1). The Ministry of the Environment installed a rain gauge for the period of June 27 to October 31, 1983 near Rockcliffe Avenue (Figure 2-1). In addition, rain gauge data from AES stations at Keele Street/Lawrence Avenue and 425 Old Weston Road was obtained for comparison purposes.

Field monitoring to measure combined sewer overflows at the Hyde Avenue tank and the Site 3 regulator was carried out by the MOE during the months of April to October 1983. Monitoring locations were established in the Hyde Avenue tank, at the overflow of the Hyde Avenue tank and the Site 3 regulators, and within a manhole 300 m upstream of the Site 3 regulators. Water levels were recorded at each of the locations in approximately five minute intervals. This information was received from the MOE and used in the calibration process of the computer model.

Precipitation data was also received from the MOE for the April to October period in 1979. This season was considered to be representative of the average season, according to the statistical analysis of the available Canadian AES precipitation data (Reference 2). The M.O.E requested that rainfall events from this period be used in analysing the existing system,

to provide results which will be comparable to previous studies carried out for the TAWMS committee.

Three storms on September 14, 1982, August 8, 1983 and August 14, 1984 were initially screened in order to select a historical event which resulted in basement flooding within the City of York. The rainfall volumes for each of these events at four stations are given in Table 2-2. The rainfall volumes for the two stations outside of York are listed to provide an indication of how widespread the storm event was.

The values in Table 2 suggest that the September 14, 1982 storm resulted in the most consistent rainfall volume within the City of York and adjoining areas. Furthermore, inspection of the flooding records shows that the most widespread flooding occurred on this date. For these reasons, the September 14, 1982 storm was selected for calibrating the computer model.

The September 14, 1982 storm, as measured at the Works Yard, lasted about three hours. During this period approximately 47 mm (1.8 in) of rainfall was measured. Analysis of this rainfall event shows that the peak one hour intensity of this storm produced 36.3 mm (1.43 in) of rain. The rainfall distribution for this one hour period is shown in Figure 2-2.

In addition to utilizing the rainfall information from the September 14, 1982 storm, it was necessary to establish design storms which could then be used in the evaluation of the existing sewer system. Several approaches have been developed which establish the required design storms. The approaches in general use the Intensity Duration Frequency (IDF) curves from a nearby weather station and assume that the rainfall for a given duration is distributed in a specific pattern over the duration of the storm. Together the rainfall volume for the given duration may be combined with the assumed rainfall distribution to produce the design storm (or rainfall hyetograph).

A widely used approach has been to apply a distribution to the IDF curve

TABLE 2-2

RAINFALL VOLUMES FOR THREE EVENTS IN 1982, 1983, 1984

Storm Date	Rainfall Gauge Location				Storm Duration (minutes)
	425 Old Weston Road	Keele and Lawrence	Rockcliffe (MOE gauge)	Castlefield Works Yard	
Sept.14/82	34.8	36.3	N/A	36.3	60
Aug. 8/83	N/A	27.4	22.3	28.5	45
Aug.14/84	40.1	47.2	N/A	45.7	85

Notes: 1) All values are depth of rainfall in (mm)
2) N/A = not available.

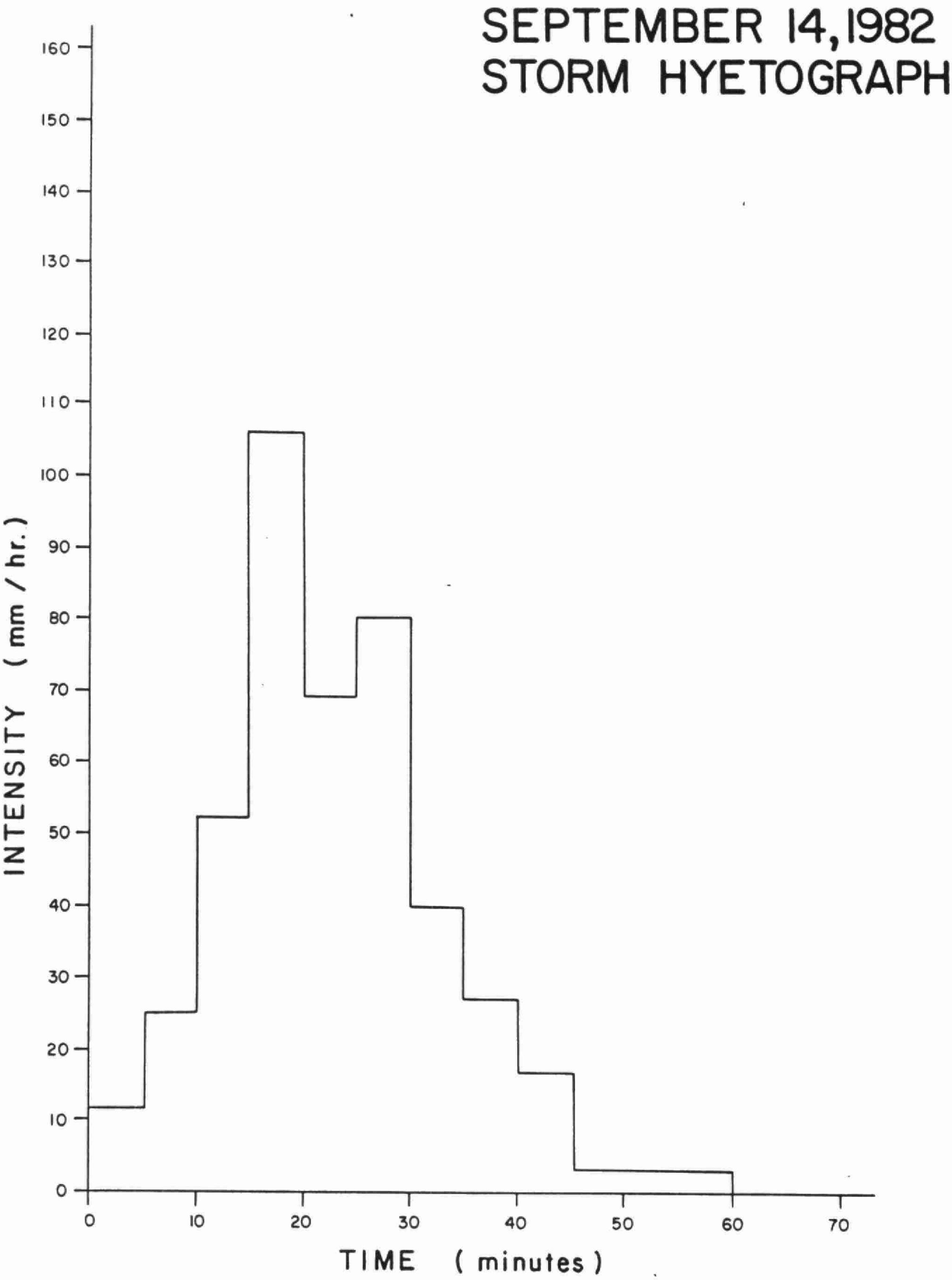

SEPTEMBER 14,1982
STORM HYETOGRAPH
INTENSITY (mm / hr.)
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
TIME (minutes)

figure 2-2

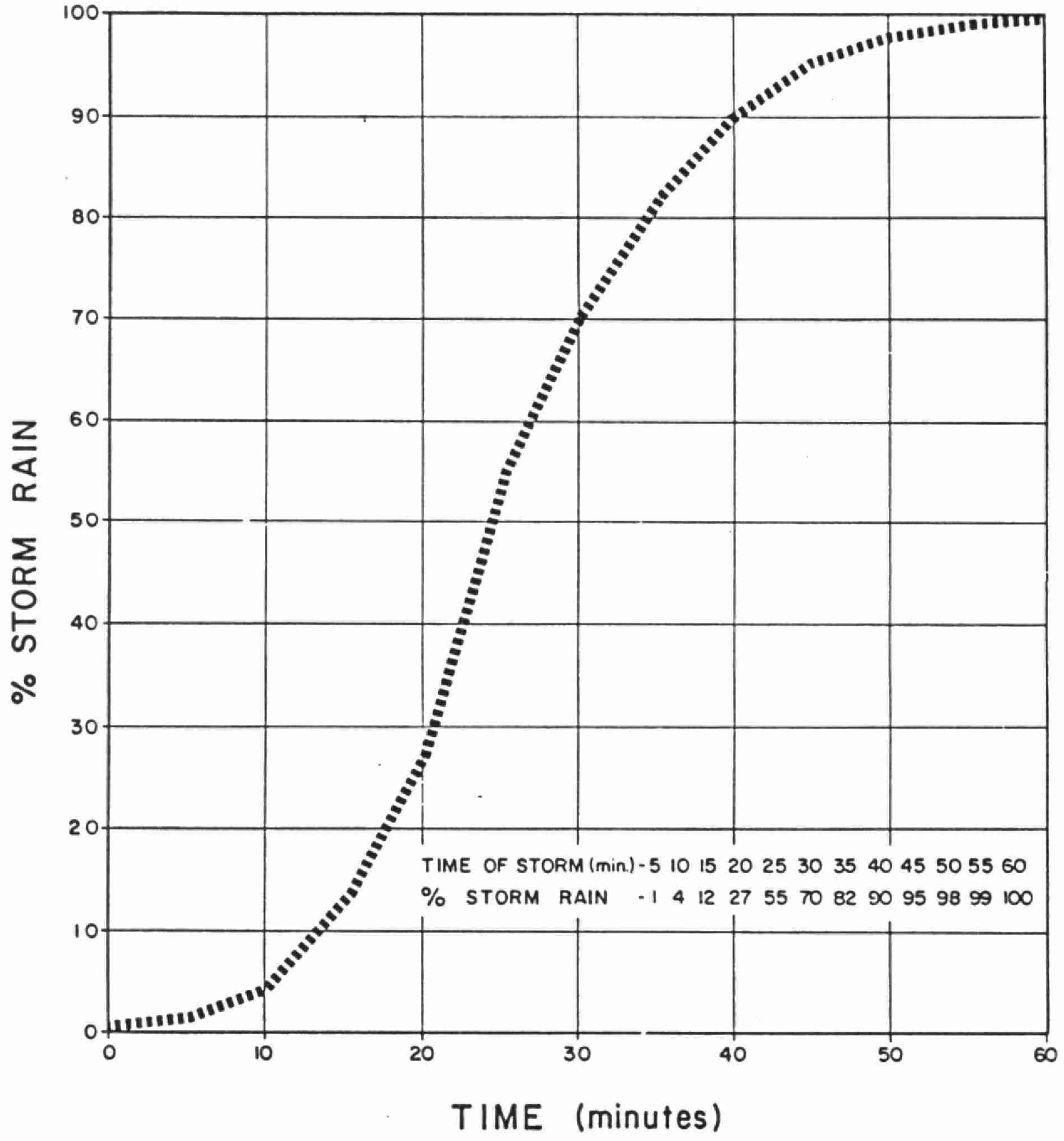
known as the Chicago Storm Distribution (Reference 3). However the IDF curves which form the basis of the Chicago Storm Distribution are derived from different storms in a time sequence other than the actual occurrence. Studies have shown that this method tends to over-estimate peak flows as compared to those generated from actual rainfall events of the same frequency. Furthermore, the distribution was developed for Chicago and is not necessary representative of storm patterns characteristic to other areas (Reference 4).

AES is studying rainfall patterns for a number of areas within Canada, one of the areas being Southern Ontario (Reference 5). After review of the study's findings and discussions with the author of the paper, it was decided to use the rainfall distribution as recommended for Southern Ontario by AES. This storm is representative of a summer thunderstorm.

The selected rainfall distribution (Figure 2-3) was then used in conjunction with the IDF curves for the City of Toronto (Figure 2-4) to establish the design storms with a 2, 5 and 10 year frequency. Several different storms durations were considered. The results from modelling carried out in previous studies by our firm for this sewer system showed that the one hour storm duration produced the most severe water levels. For this reason a one hour storm duration was selected. The rainfall hyetographs for the three storms are illustrated in Figure 2-5.

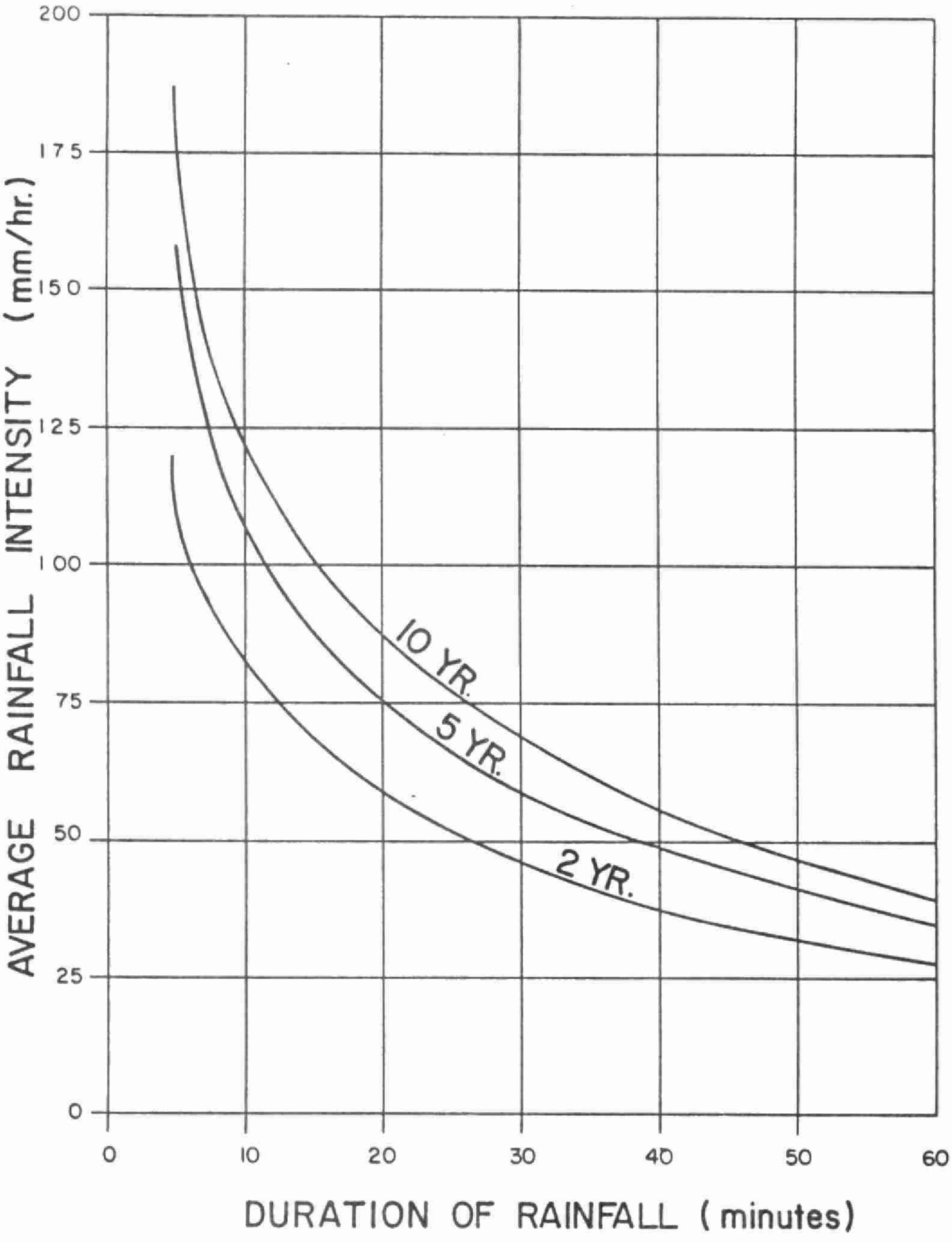
The flow monitoring data from the MOE and the precipitation records from the Castlefield and Rockcliffe rain gauges were reviewed in order to select a storm event to calibrate the model for combined sewer overflows. Upon inspection of the available data it was decided to use the October 12, 1983 storm to calibrate the computer model. The reasons for selecting this storm are as follows:

i) both the Rockcliffe and Castlefield rain gauges were in operation for this storm event. Furthermore, the total rainfall volumes at each station compare favourably (21.0 mm at Rockcliffe vs 18.2 mm at Castlefield).



SOUTHERN ONTARIO
AES - 1 HOUR STORM
TIME PROBABILITY
RAIN DISTRIBUTION

figure 2-3



INTENSITY - DURATION FREQUENCY CURVES

figure 2-4

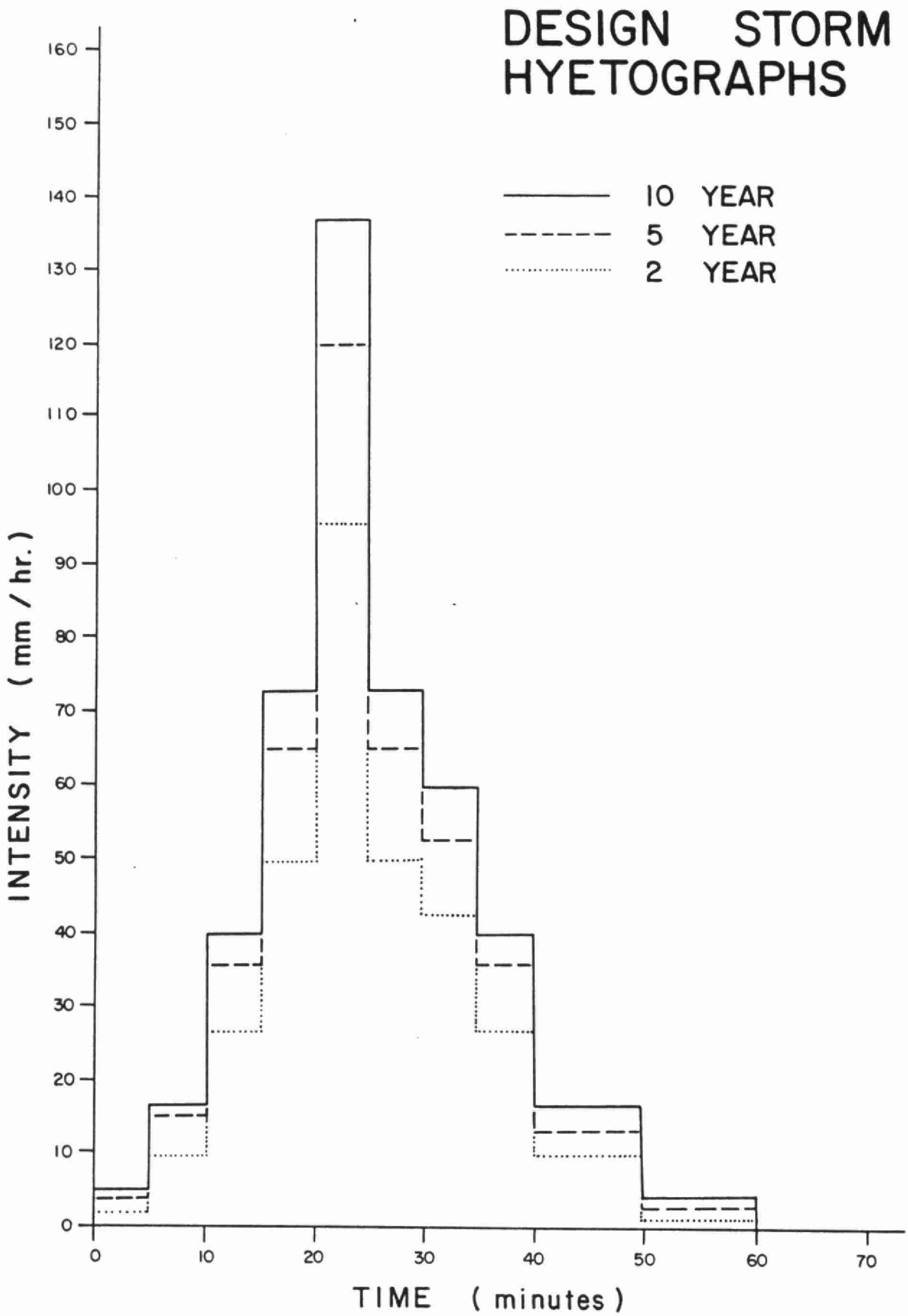

DESIGN STORM
HYETOGRAPHS
10 YEAR
5 YEAR
2 YEAR
INTENSITY (mm / hr.)
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
0
10
20
30
40
50
60
70
TIME (minutes)

figure 2-5

ii) the event was large enough to cause overflows at both the Hyde Avenue tank and the Site 3 regulator.

The rainfall distribution which was used in calibrating the computer model was based on the average of the rainfall volumes at the Rockcliffe and Castlefield rain gauges. The October 12, 1983 storm lasted about ten hours during which an average volume of 19.6 mm was measured. The rainfall distribution for the storm event is shown in Figure 2-6.

In addition to selecting a rainfall event for calibration, a series of five storm events during the months of April to October 1979 were selected. These events were used to estimate the combined sewer overflow volumes at Hyde Avenue and the Site 3 regulator under existing conditions and with the remedial works in place. The five storm events which were selected are listed in Table 2-3 as are the maximum one and two hour volumes, the total rainfall volumes and the antecedant dry period preceeding each event. Figures 2-6 and 2-7 illustrate the rainfall distribution for each event. The rationale for selecting the five events is given in Section 4.2.

TABLE 2-3

RAINFALL VOLUMES FOR FIVE 1979 STORM EVENTS

Storm Date	Maximum 1 Hour Volume (mm)	Maximum 2 Hour Volume (mm)	Total Rainfall Volume (mm)	Antecedent Dry Period (days)
May 3	4.3	6.1	12.7	3
June 10	16.4	16.4	17.4	3
July 11	36.7	37.3	37.3	1
August 2	9.2	12.1	14.3	2
Sept. 14	5.3	8.3	34.6	4

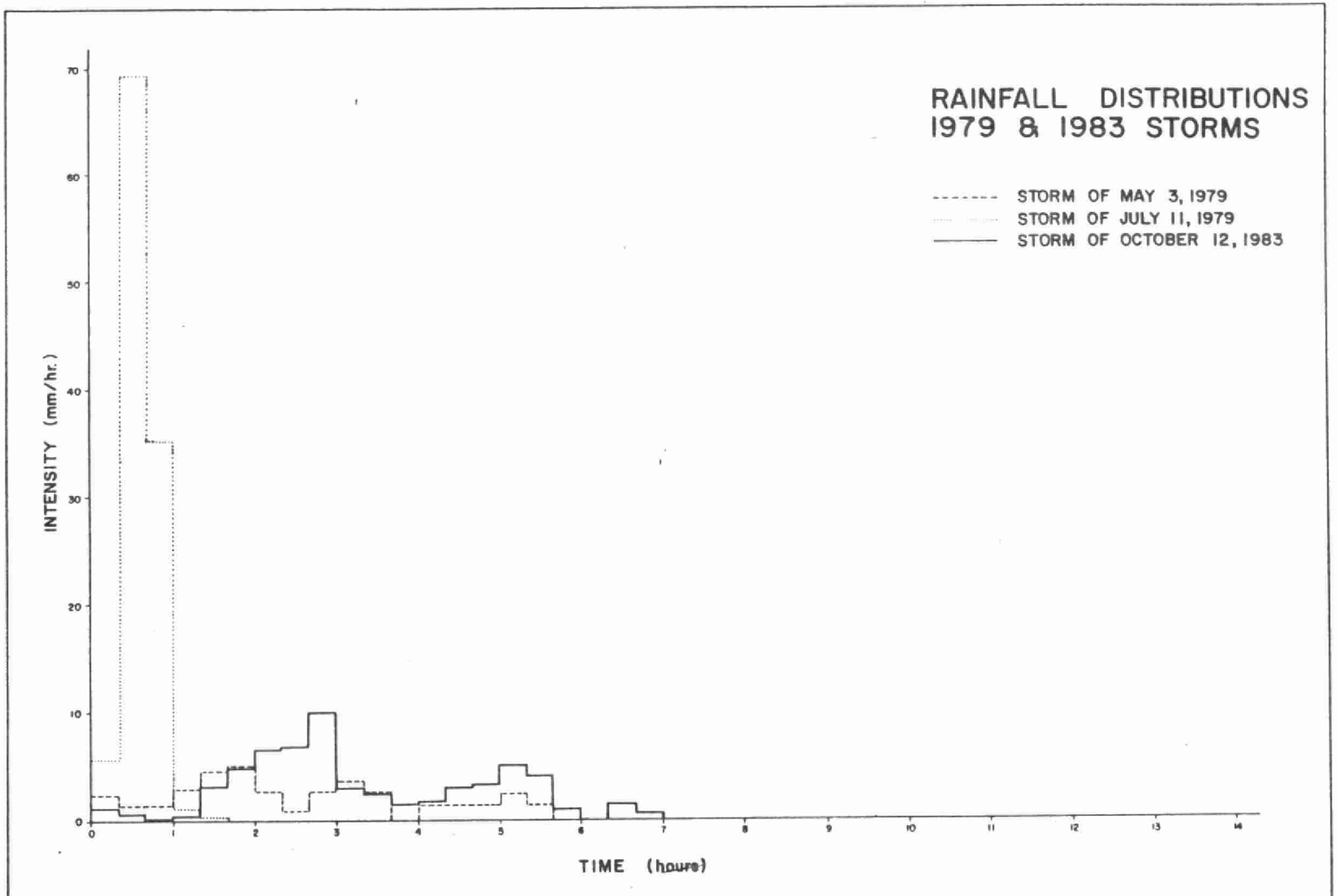

RAINFALL DISTRIBUTIONS
1979 & 1983 STORMS
STORM OF MAY 3, 1979
STORM OF JULY 11, 1979
STORM OF OCTOBER 12, 1983
INTENSITY (mm/hr.)
TIME (hours)

figure 2-6

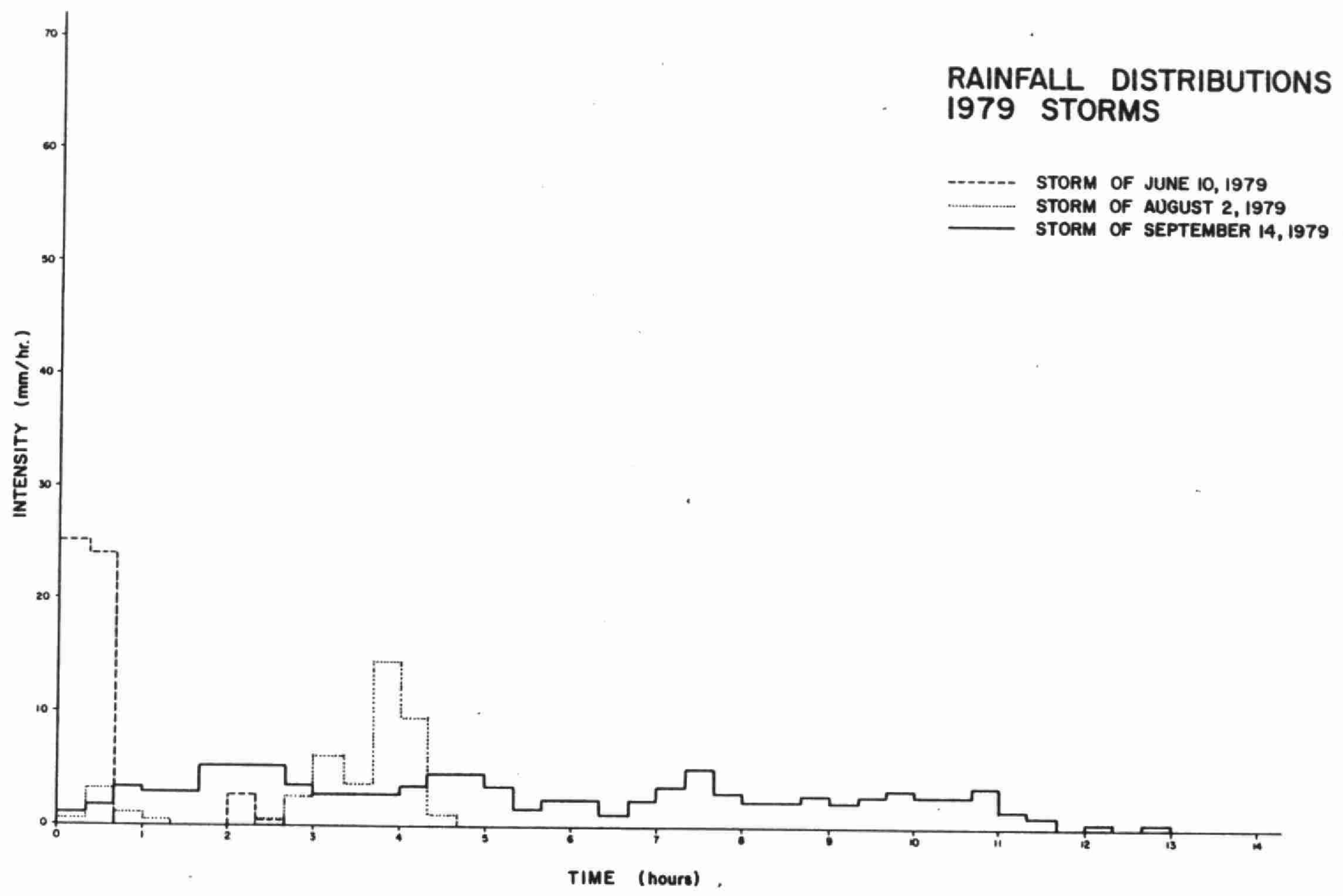

RAINFALL DISTRIBUTIONS
1979 STORMS
STORM OF JUNE 10, 1979
STORM OF AUGUST 2, 1979
STORM OF SEPTEMBER 14, 1979
INTENSITY (mm/hr.)
TIME (hours)
0
10
20
30
40
50
60
70
1
2
3
4
5
6
7
8
9
10
11
12
13
14

figure 2-7

3 - MODEL PREPARATION

3.1 Model Selection

The computer model best suited for an analysis of this type must be capable of simulating hydraulically complex sewer networks, while at the same time accurately predicting storm runoff from various land uses. Furthermore, due to the size of the study area, the model must be able to handle a large number of pipes in a single simulation. The required simulation capabilities are as follows:

1. Unsteady characteristics of flow processes.

2. Individual runoff hydrographs developed for roof, grassed and paved areas within each sub-catchment.

3. Retention capacity of the watershed and sewer network.

4. Flow-regulating effect of backwater.

5. Interconnective effect in a network generated at sewer branches.

6. Surcharged conditions within the sewer network.

7. Overflow weirs, bypass structures, retention basins etc.

8. Storm input data in the form of block design storms, synthetic model storms as well as actual rainfall events.

9. Generation of dry weather flow characteristics including domestic and industrial sewage flows and infiltration into the sewers.

The model selected for this study is called the Hydrograph Volume Method

(HVM) which was developed by Dorsch Consult in West Germany (Reference 6). The model has received extensive testing and has been employed for several studies in Ontario, Vancouver, Regina and Cleveland. Furthermore, the model has been used over the past ten years by the City of Toronto. HVM is a single event model which uses an implicit solution for solving the complete St. Venant equations for up to 1350 pipes. This type of solution, although computationally expensive, should ensure that the model will reliably simulate the flow conditions within the sewer system even under extreme surcharging conditions. The HVM model has been used as a benchmark by the developers of the Extended Transport Block to compare the accuracy of their model (Reference 7, 8).

Our firm has used the HVM in several studies over the past nine years, including studies completed for the City of York where a total of about 800 sewer segments have been modelled.

3.2 Model Description

The HVM model is comprised of three separate models and an editing routine which checks for user input errors. The first model (Surface Runoff Model) generates runoff hydrographs for various surface characteristics which are encountered in a drainage basin. The input parameters to this model include a design or historically recorded rainfall event, parameters describing overland flow lengths, roughness co-efficients for grassed, paved and roof areas, basin slopes, infiltration parameters for the Horton Equation and dry weather flow input including domestic and industrial sewage flows and infiltration rates. The output is comprised of a runoff hydrograph for the pervious area and a combined runoff hydrograph for the impervious (street and roof) areas. The hydrographs are generated for each surface code. For this study, a total of 16 separate surface codes representing the various land uses were developed. The surface codes include various types of residential areas, commercial lands, industrial lands, open spaces, schools and apartments.

The second model (Partial Fill Model) generates values of velocity and flows for partially filled sewer segments of various shapes. Pipe shapes representing circular sewer pipes, road cross sections, horseshoe shaped pipes and box culverts were generated for this study.

The third model (Sewer Network Flow Model) uses the input from the first two models to generate the composite sub-basin hydrographs for each sewer segment. These hydrographs are in turn routed through the system to establish the peak flow rates and water levels in each pipe segment. The input to this model includes for each pipe segment the size, length, upstream and downstream inverts and ground elevations, roughness coefficient, tributary area, percent imperviousness of the tributary area, the dry weather flow information and the pipe interconnections.

The output from the model is extensive and includes for each pipe segment the peak flow and water level, the time varying water levels and flows, dry weather flows, loading ratios (the value of the peak flow in the pipe divided by the design capacity of the pipe) and the difference in elevation between the maximum water level and the surface elevation.

4 - EVALUATION OF THE EXISTING SYSTEM

4.1 Calibration Process

The HVM model is a deterministic model, that is to say if the input parameters are representative of the study area, the model will closely simulate the hydraulic response of the system. Field surveys, assessment maps, topographic maps and sewer profiles were used to establish many of the model parameters. In order to ensure that the model would produce results representative of the actual system responses, a calibration procedure was undertaken using data collected from the City of York for the September 14, 1982 storm and from the MOE for the October 12, 1983 storm. The objective of the calibration process was to establish a computer model which is consistent with the level of study i.e. a model which would provide reasonable results at the planning or feasibility stage.

The first step in the calibration process was to define the existing conditions to be the conditions that existed as of the end of 1982. The total catchment area is 690 ha, of which 89 percent is serviced by combined sewers. The flooding records obtained from the City of York together with the flow monitoring results from the MOE were used as a basis to calibrate the model for the above two storms.

The calibration procedure for the combined sewer system involved a series of simulations, varying model parameters until a reasonable correlation between the simulated water levels and the water levels as established from the available data was achieved. For the September 14, 1982 storm, the final calibration run produced maximum water levels within the individual pipe segments which were generally consistent with the available flood records. Furthermore, the peak flow rates and water levels were quite similar to values as determined in a previous more detailed study carried out by our firm in the Silverthorn Area (a comparison of the peak flows at several locations showed an average difference of 4.8 percent) (Reference 9).

For the October 12 1983 storm, the data received from the MOE consisted of water levels (in five minute time increments) in the Hyde Avenue tank, at the Hyde Avenue overflow weir, at the Site 3 overflow weir, and at a monitoring location 300 m upstream of Site 3. The maximum water levels at the two overflow sites were converted to flow rates using the appropriate broad crested weir equation for each overflow (Reference 10). To obtain the flow rates into the Hyde Avenue tank, and the volume of inflow to the tank, the water level data had to be integrated and the volume of overflow at the Hyde Avenue tank had to be taken into account. The recorded water levels at the manhole upstream of Site 3 were compared directly with the simulated values.

A comparison of the results from the calibration run to the calculated values using the field monitoring data is given in Table 2-4. The results from the calibration compare reasonably well with the field monitoring data. The calibrated results are, however, conservative in that the overflow volumes at Hyde Avenue and Site 3 are higher than the values recorded in the field.

Table 2-5 lists several of the parameters which were used to calibrate the model.

4.2 Existing Combined Sewer System

The computer model which was used in the calibration process was updated to reflect conditions as they are likely to be at the end of 1985. During the 1983 to 1985 period storm sewer separation has been carried out in two areas. A storm trunk sewer which flows southeasterly to the City of Toronto has been constructed along Locksley Avenue, Eglinton Avenue, Oakwood Avenue and Clovelly Avenue. An additional storm trunk sewer which flows westerly along Rotherham Avenue to the Black Creek has also been constructed (see Plate 3-2). Consistent with other areas within the City of York which have been separated, the storm sewers convey runoff only from the grassed and paved areas. The roof areas which are directly connected

TABLE 2-4

SIMULATED VS MONITORED VALUES FOR THE OCTOBER 12., 1983 STORM

Parameters	Value from Field Monitoring	Value from Computer Simulation
Peak Flow Rate at Overflow Weir, Hyde Ave (cms)	1.27	1.20
Total Overflow Volume at Hyde Ave. (m^3)	8,675	11,606
Total Inflow Volume to Hyde Ave (m^3)	16,075	18,900
Peak Flow Rate at Overflow Weir, Site 3 (cms)	0.13	0.78
Total Overflow Volume at Site 3 (m^3)	540	3,100
Maximum Water Level in Manhole Upstream of Site 3 (m)	1.00	0.81

TABLE 2-5

HVM MODEL PARAMETERS

Land Use Features	Grass	Paved	Roof
Ground Roughness	0.33	0.25	0.0117
Ground Slope (m/m)	0.01	0.01	0.01
Average Inlet Width (m)			
Residential	38.00	80.00	34.00
Commercial	240.00	270.00	200.00
Industrial	260.00	305.00	250.00
Infiltration Rate (l/s/ha)			
Inital	120.00	-	-
Final	20.00	-	-
Decay Coefficient	5.00	1.00	1.00
Depression Depth (mm)	5.00	1.00	1.00

General Parameters

No. of Sub-catchments	295
Percentage Roof Downspouts Connected to Combined Sewer (%)	86
Average Percent Impervious of Total Catchment Area (%)	0.58
Average Mannings n Value	0.015
Domestic Sewage Inflow (l/cap/day)	454
Average Population Density (persons/ha)	57
Dry Weather Infiltration (l/s/ha)	0.10
Time Increment September 14, 1982 storm (min)	5.00
Time Increment October 12, 1983 storm (min)	15.00

to the combined sewer are not disconnected. Together, the construction of the two trunk storm sewers and the associated laterals has reduced the drainage area to the Hillary Avenue system by about 60 ha. The total drainage to the Hillary Avenue combined sewer system will therefore be approximately 630 ha at the end of 1985.

4.3 Basement Flooding for Design Storms

To estimate the degree of flooding within the Hillary Avenue sewer system, the existing (1985) sewer system was simulated and analyzed for the 2, 5 and 10 year storms. The resulting backwater levels within the study area are shown schematically for the 2 year storm (Plate 2-2) , the 5 year storm (Plate 3) and the 10 year storm (Plate 2-4). The backwater levels as shown on the Plates illustrate which areas are subject to basement and street flooding for each of the design storms. It should be recognized that due to the relief afforded the combined sewer system through the flooding of basements and the varying basement floor elevations within any one pipe segment,only the lower lying basements within the areas designated as flood susceptible are likely to experience basement flooding. In estimating which areas would be susceptible to basement flooding, an average basement floor elevation of 1.5 m below the centre line of the street was used. This value was determined from numerous profile drawings.

Peak flow rates at several locations within the study area are given in Table 2-6. The flow rates are listed for each of the 2, 5 and 10 year storms. Pipe capacities of the existing combined sewers are also given.

The maximum backwater levels within the study area for the 2 year storm indicate that basement flooding will occur in eight localized areas. Based on the field surveys and the computer analysis, the flooding problems would seem to be due to undersized laterals and isolated low lying areas. The basement flooding problems along McRoberts Avenue are, however, attributable to the backwater effects from the combined sewer along Dunraven Drive.

TABLE 2-6

PEAK FLOWS FOR THE 2, 5 AND 10 YEAR DESIGN STORMS

Location	Pipe Segment No.	Storm Event			Pipe Capacity
		2 Year	5 Year	10 Year	
Times Rd.	S10	2562	3644	4340	4790
Easement	S22	1290	1693	1835	1918
Strathearn Rd.	S46	3229	4408	5199	3283
Rogers Rd.	S59	3071	3981	4784	10147
Easement	S71	4858	5811	7040	2649
Dunraven Dr.	S135	6275	9018	10473	9524
Dunraven Dr.	S136	6359	6267	6045	5889
Rogers Rd.	S184	1290	1604	2062	1911
Rogers Rd.	S185	10797	12595	14437	16400
Hyde Ave.	S190	15436	16511	21570	12594
Hyde Ave.	S192	6619	9253	11471	10992

Notes: 1. All flows are in litres per second.
2. Pipe Segment numbers are shown on Plate 2-1.

For the 5 year storm the flooding problems are more widespread and surface ponding will occur in many low lying areas. The basement flooding problems are due in part to undersized laterals, isolated low lying areas, and in several situations to undersized trunk sewers. The trunk sewers along Glencedar Road, Chudleigh Road-Dunraven Drive, and Atlas Avenue are all overloaded and,in turn, increase the water levels in the adjacent laterals.

The 10 year design storm will result in basement flooding within roughly 25 percent of the study area. This percentage is rather low when compared to other cities which have a history of basement flooding problems. It should, however, be kept in mind that previous remedial works in the form of storm sewer separation and inlet control/detention tank storage have been constructed within the study area. Furthermore sections of the study area have rugged terrain and flooding problems within these areas tend to occur only in the isolated low lying areas.

As for the 5 year design storm, the flooding problems are due in part to undersized trunk sewers, isolated low lying areas and undersized laterals. Undersized trunk sewers which cause backwater problems within adjacent laterals are located along Times Avenue, Glencedar Road, Atlas Avenue, Holmesdale Road-Chudleigh Road-Dunraven Drive (1350 mm and 1650 mm) and Rogers Road (1500 mm x 1800 mm).

4.4 Combined Sewer Overflows for 1979 Storm Events

To estimate the frequency and magnitude of the combined sewer overflows five historical storm events which occurred during the April-October 1979 period were selected. The HVM model is a design event model. Therefore it is not possible to analyze on a continuous basis all the storm events for the April-October months. However, due to the nature of this study, where both combined sewer overflows and basement floodings within a relatively large study area had to be addressed, it is unlikely that continuous modelling of all the storm events would have been practical. The level of discretization (295 pipe segments were modelled) necessary to properly

account for all the flow diversions, significant hydraulic features and backwater effects which influence the degree of basement flooding, combined with the relatively short time step used (5 minutes for basement flooding events and 20 minutes for combined sewer overflow events) would have resulted in continuous modelling being prohibitively expensive.

Prior to the selection of the five storm events, the precipitation data for the April to October period was analyzed. Each of the 25 events with total rainfall volume in excess of 6.0 mm was ranked in accordance to the total precipitation which occurred during each event. The largest storm received a ranking of 1, the second largest 2, etc.). Five storm events which covered a broad range (from an event where the total precipitation volume was exceeded by ten other events to an event with approximately a 3 year return frequency) were selected. Total precipitation volume was selected as the governing criteria as the Hyde Avenue tank has no outlet to the sanitary trunk sewer during storm events, and this regulator contributes the largest percentage of combined sewer overflow to the Black Creek. A second criteria for selecting the storm events was the antecedant dry periods. In the computer simulation, it was assumed that the Hyde Avenue tank was empty at the beginning of the storm event. It was therefore attempted to select storm events with long antecedant dry periods.

The results from the simulation for the five events are given in Table 2-7. The results show a clear relationship between the total precipitation volume and the total combined sewer overflow volume at the two regulators. Also included in Table 2-7 are the overflow volumes as measured for the October 12, 1983 storm using 1982 and 1985 conditions. The results indicate that disconnection of the grass and paved areas for 14 percent of the catchment area (due to the storm sewer separation that was carried out during this three year period) results in a 29 percent reduction in combined sewer overflow. The total area of grass and paved area disconnected from the combined sewer is approximately 60 ha.

TABLE 2-7

COMBINED SEWER OVERFLOW VOLUMES

Storm Date	Storm Ranking	Total Rainfall Volume (mm)	Overflow Volume Hyde Ave (m^3)	Overflow Volume Site 3 (m^3)	Total Overflow Volume (m^3)
May 3, 1979	11	12.7	0	0	0
August 2, 1979	9	14.3	3,900	2,200	6,100
June 10, 1979	8	17.4	11,600	3,100	14,700
Sept. 14, 1979	2	34.6	19,400	1,200	20,600
July 11, 1979	1	37.3	67,700	11,400	79,100
(1)Oct.12,1983	-	19.7	11,600	3,100	14,700
(2)Oct.12,1983	-	19.7	8,400	1,900	10,300

Notes: 1) For the October 12, 1983 storm event; (1) represents the 1982 conditions and (2) represents 1985 conditions.

5 - GENERATION AND ANALYSIS OF REMEDIAL WORK ALTERNATIVES

Several alternative remedial solutions may be implemented in order to reduce both basement flooding problems and the frequency and/or magnitude of combined sewer overflows to the Black Creek. The methodology that was used to solve these problems is outlined below:

1. The remedial works required to eliminate basement flooding for the 10 year design storm within the Hillary Avenue sewer system were established.

2. With the above works in place, it was then determined what additional remedial works would be required to eliminate combined sewer overflows at Hyde Avenue and Site 3 for each of the five storm events in 1979.

3. For the Hillary Avenue sewer system, the remedial works required to eliminate basement flooding for the 2 and 5 year design storms were then established.

4. Using available data, the results from the Hillary Avenue sewer system were then extrapolated to establish the remedial works necessary for the remaining areas within the City of York which are serviced by combined sewers (primarily the Mt. Dennis and Rockcliffe catchment areas).

As stated in the Terms of Reference, the principal component of the remedial works is to be detention tank storage. The feasibility of providing local storm sewers was considered, however due to the number of trunk sewers which are surcharged for the larger design storms and the associated cost of installing storm sewers (see Task 3) the recommended remedial works include the construction of only a few storm sewers.

In order to reduce basement flooding,four different methods were evaluated.The first two methods, which are used for the combined sewer laterals, involve restricting the inflow to the existing sewer system and storing the excess runoff in underground detention tanks. These methods were considered in light of the fact that existing sewers are generally adequate except during periods of intensive rainfall. It may therefore be cost efficient to restrict the rate of inflow to the existing sewers to a level which does not cause basement flooding. We have termed this method the INLET CONTROL METHOD.

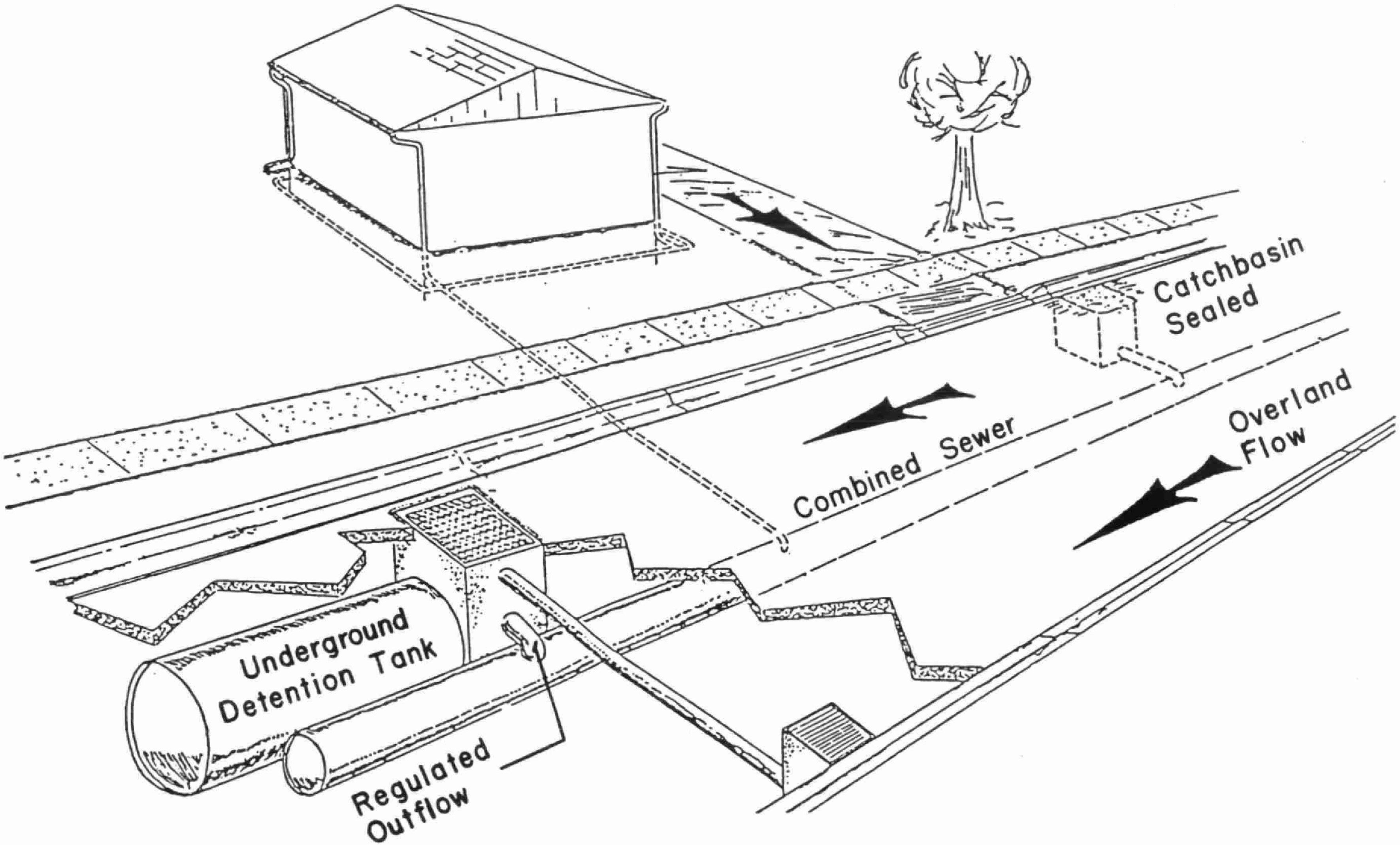
The third method is to alleviate the surcharge levels within a combined trunk sewer by providing an overflow to an underground storage facility.

The fourth method for reducing basement flooding involves installing storm sewers. Runoff from grass and paved areas would be redirected to the storm sewers, thereby reducing the inflow rate to the existing combined sewers.

The two types of inlet control proposed for this study are explained in more detail below.

The first method of inlet control, as described above, involves restricting the rate of inflow to the sewer so that the capacity of the existing combined sewer system is not exceeded. The excess runoff will be stored in underground tanks within the public-right-of way. The stored runoff will be released back into the existing combined system at a controlled release rate. To limit the inflow to the sewers, a percentage of the catch basins will be sealed and the flow from the grassed and paved areas will be directed overland to the underground storage tanks. At other catch basins, particularly those located in low lying areas, restrictors which limit the inflow to a specified rate will be installed. Figure 2-8 illustrates the basic concept for this type of inlet control.

A diagram illustrating one method for installing a catch basin restrictor



UNDERGROUND DETENTION SYSTEM

figure 2-8

is given in Figure 2-9. The restrictor is bolted to the catch basin top. This method of installation should ensure that the restrictor will not inadvertently be removed. The opening at the middle of the restrictor controls the inflow to the sewer system. The restrictor may be removed with the catch basin top to clean any debris that builds up. The opening has been sized so that debris may be cleaned out by hand. A 90 mm (3-1/2 in) opening has been selected for this study. Furthermore, it has been assumed that, on average, three catch basins for every 2 ha of drainage area will have catch basin restrictors installed. The remaining catch basins will be sealed.

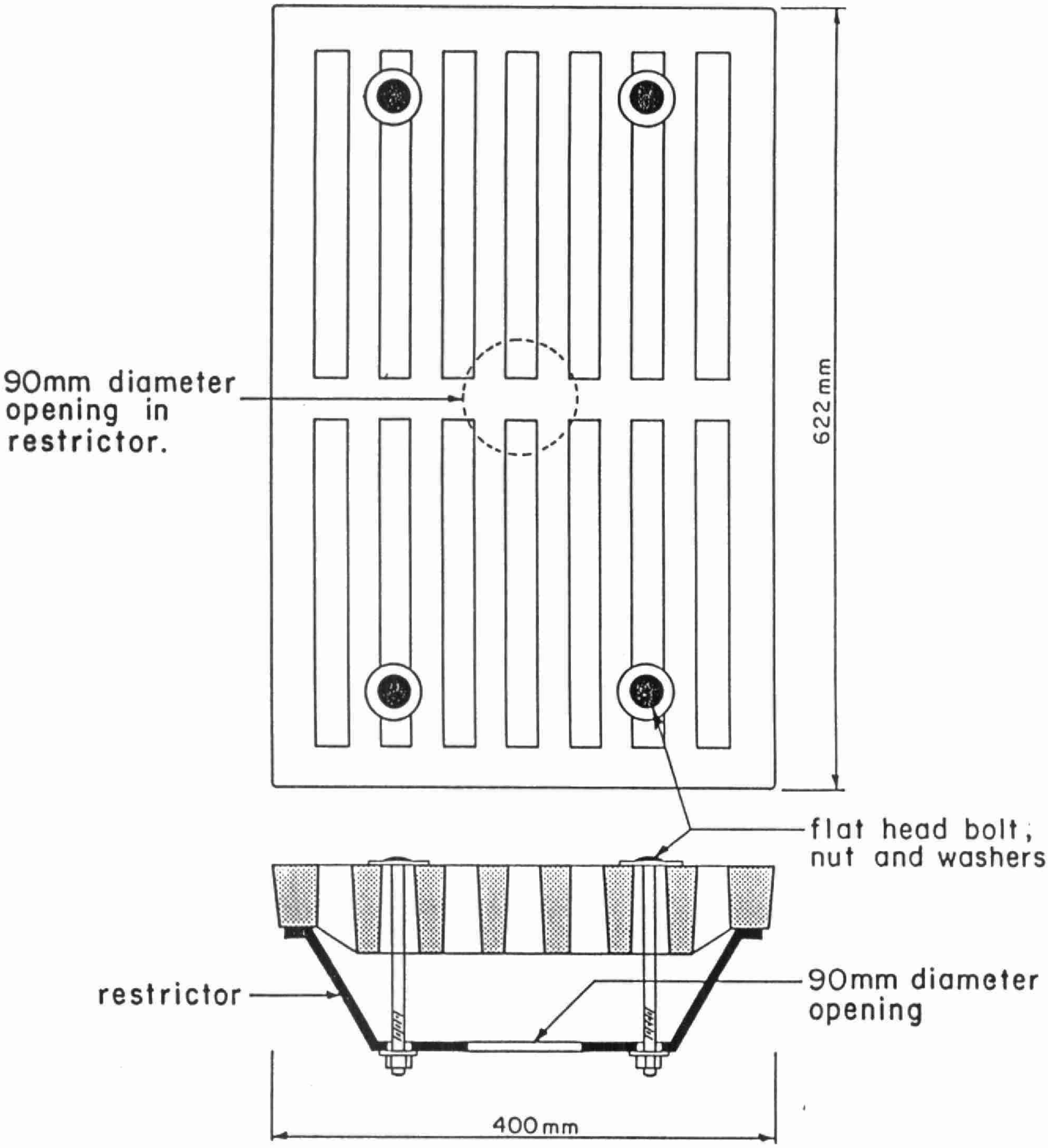
A typical method for sealing a catch basin is shown in Figure 2-10. This method has been used by our company in previous projects and was found to be successful. The existing frame and grate are removed. Clear stone is used to fill the catch basin and a wire mesh is used to cover the catch basin lead. A concrete cap is installed. In addition, the adjacent length of curb will likely have to be replaced and asphalt to fill in the low lying area around the catch basin may be required.

There are less costly methods of sealing a catch basin. Two such methods are described below.

A catch basin restrictor similar to the restrictor shown in Figure 2-9 could be installed. A small 35 mm (1-1/2 in) opening which would drain only the immediate area would be used.

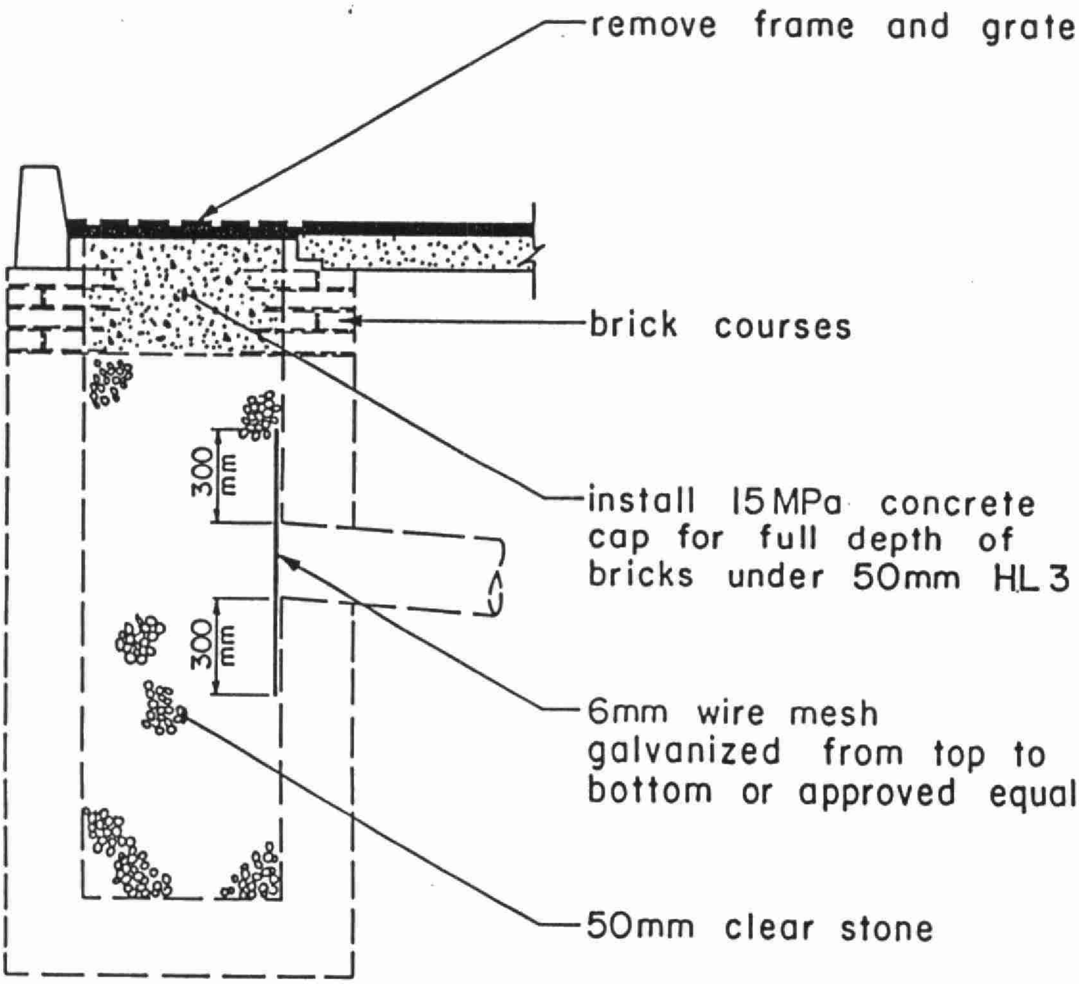
Alternatively, the catch basin could be filled to the top with clear stone. A filter cloth with a low permeability coeffficient would be placed between the catch basin top and the clear stone.

Both of the above methods of catch basin sealing will provide drainage of adjacent areas. Furthermore, a catch basin which is not totally sealed may be advantageous during the spring break-up period when it is necessary to drain local areas. It may, however, be necessary to provide periodic maintenance if one of the above two methods is employed.



CATCH BASIN RESTRICTOR

figure 2-9



TYPICAL DETAIL FOR SEALING EXISTING CATCH BASINS

figure 2-10

The head regulating chamber (Figure 2-11) for the underground tanks is designed to minimize the tank size. The water level within the chamber will rise quickly, thereby permitting the maximum release rate to be attained. Outflow from the tank to the combined sewer system will occur only when the water level in the tank exceeds the level in the chamber.

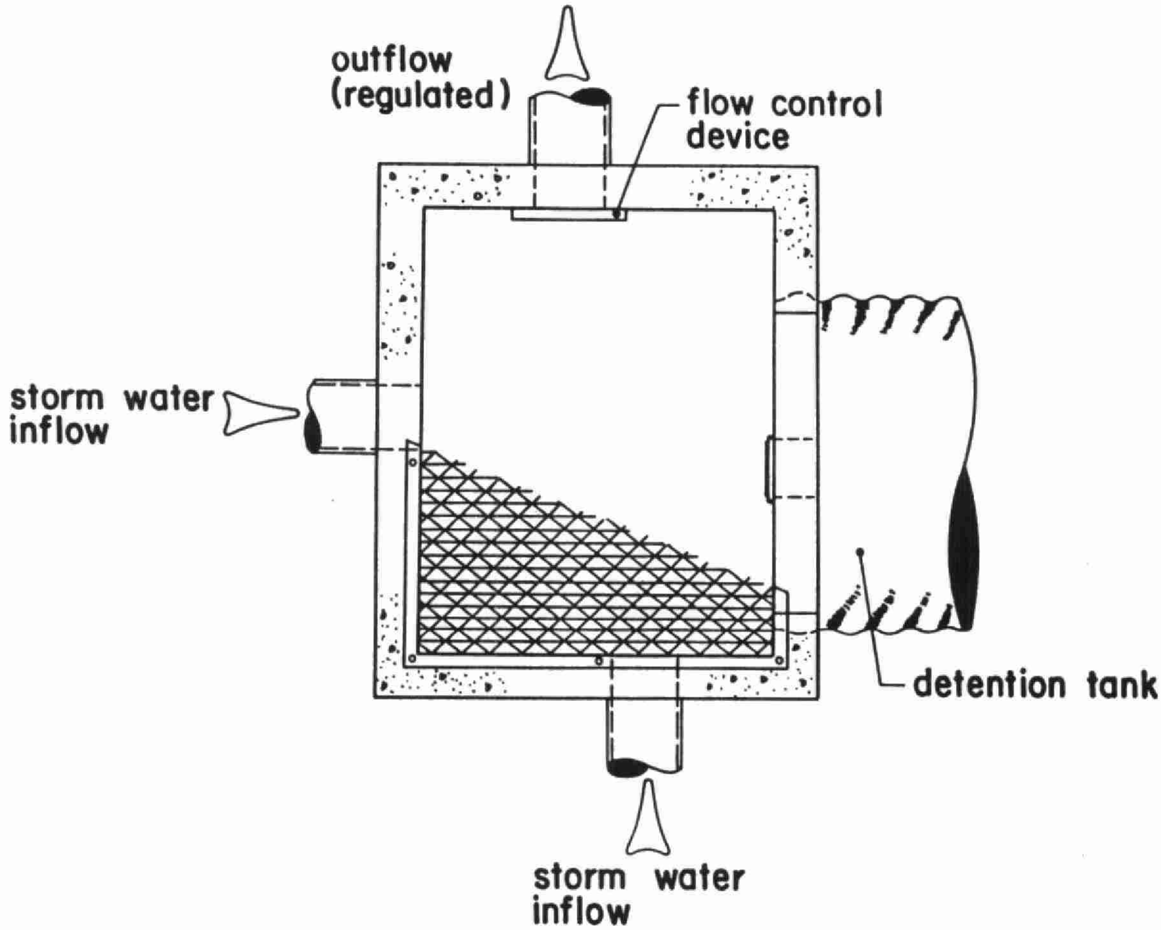
The second method of inlet control proposed is to disconnect the house connections for homes which are flood susceptible from the existing combined sewers. The house connection, to which the sanitary service, foundation drain and roof downspout are connected, will be reconnected to a separate underground storage tank. The underground tanks will store the water until the storm flows subside. The water will then be released to the existing combined sewer system. A backflow valve will prevent water from the combined sewer system from entering the underground tank (Figure 2-12).

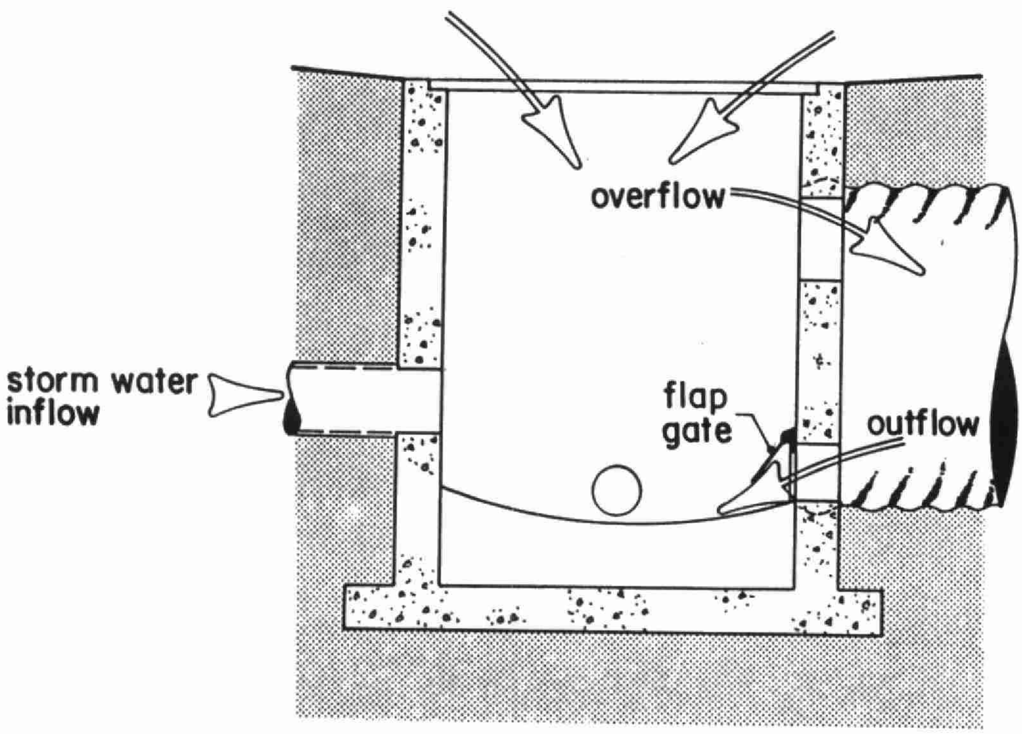
Storage of excess inflow in a large underground storage facility is beneficial in that it will reduce the surcharge levels within the trunk sewer and adjacent laterals thereby reducing the amount of remedial works that would have to be constructed in other areas. Furthermore, if this facility can be constructed within an existing park, the construction costs should be quite reasonable.

Figure 2-13 shows schematically how the underground storage facility would be incorporated for this study. Water would flow into the storage area as the water level within the combined sewer approached the obvert. This water would be stored temporarily until the water levels within the existing system have subsided. The backflow valve would then open, releasing the water from the underground storage facility. On average, the storage facility would be used to store excess water once every year.

5.1 Recommended Remedial Works for Basement Flooding - 10 Year Design Storm

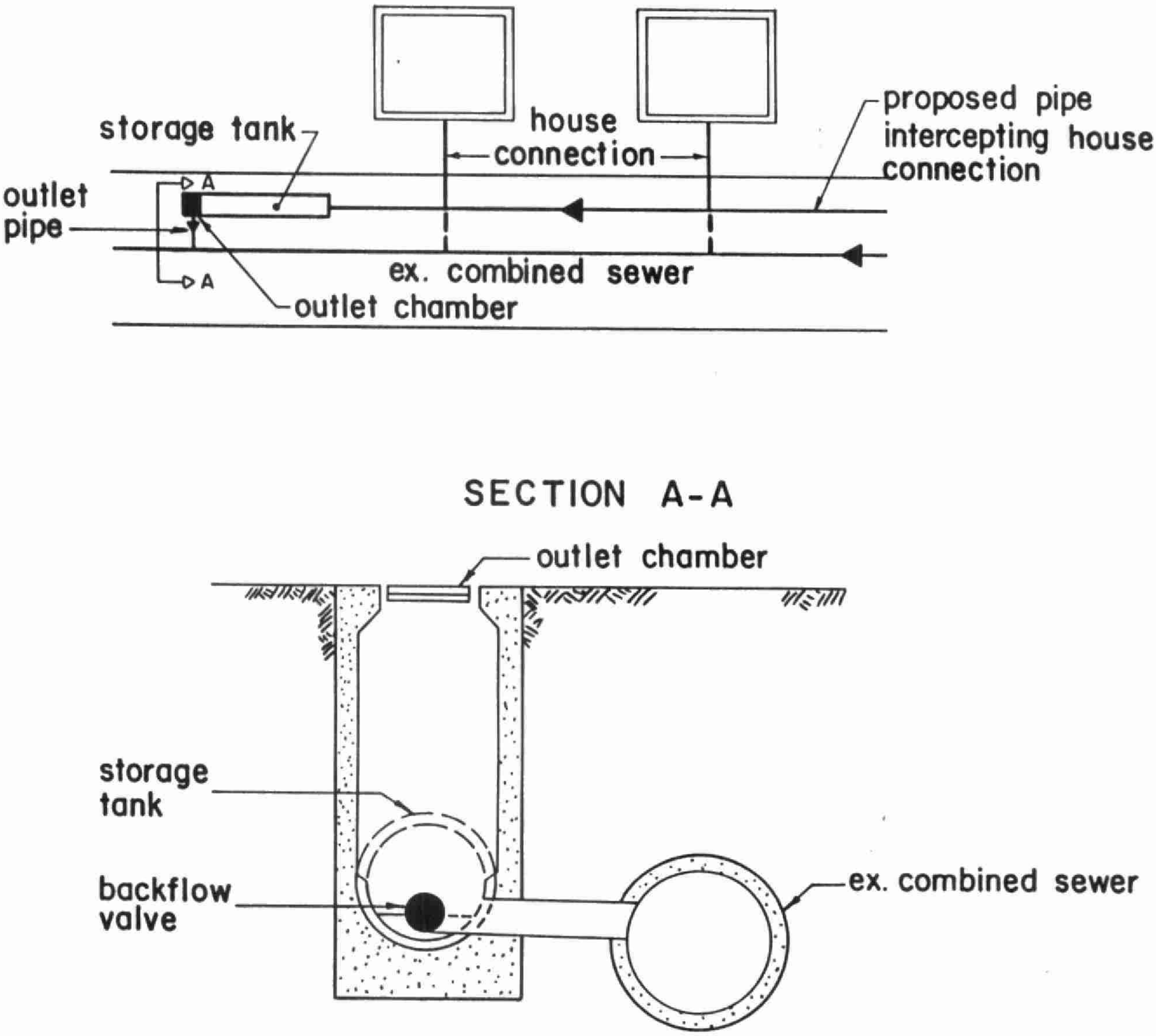
The recommended solution for providing a 10 year level of protection





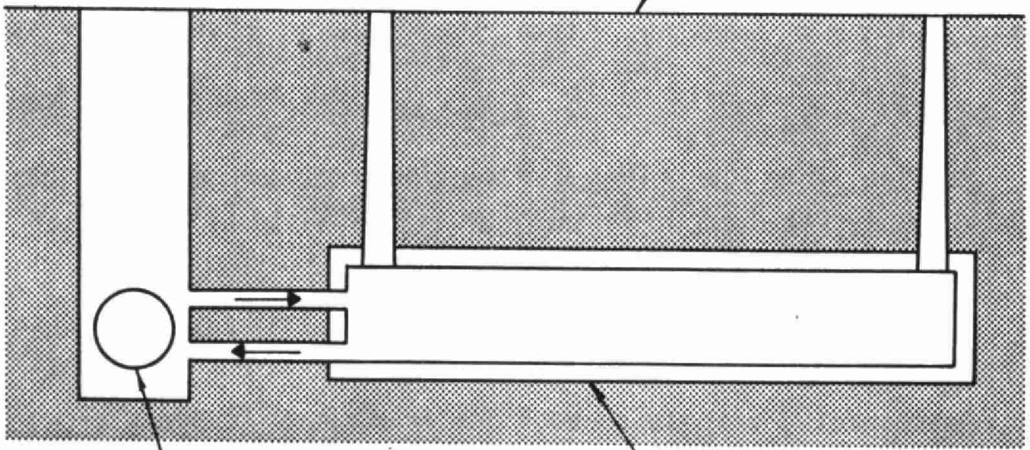
HEAD REGULATING CHAMBER

figure 2-11

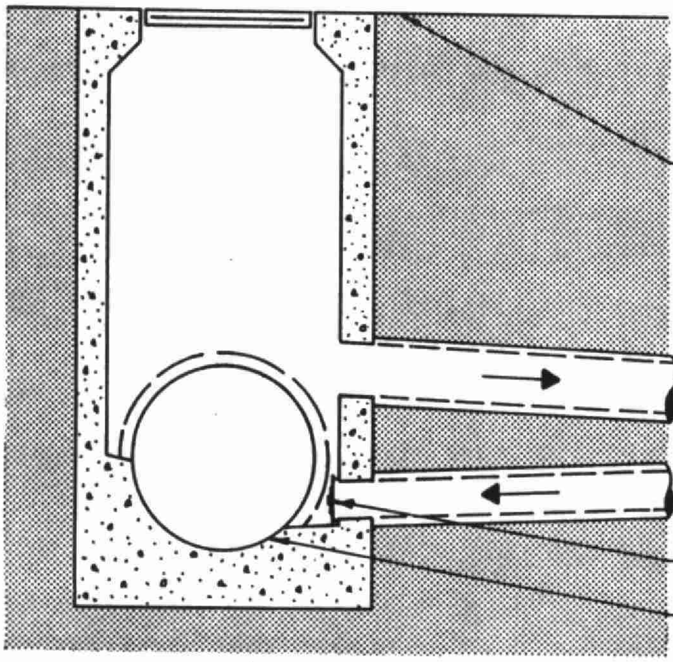


UNDERGROUND STORAGE FACILITY FOR HOUSE CONNECTION FLOWS

figure 2-12



PROPOSED OVERFLOW STRUCTURE



UNDERGROUND STORAGE FACILITY

figure 2-13

against basement flooding within the Hillary Avenue sewer system is outlined below:

1. Provide inlet control and detention tank storage for the areas as shown in Plate 2-5. A total of 115 ha would be controlled.

2. Construct a 4,000 m^3 underground storage facility in Caledonia Park.

3. Disconnect the house connections from about 40 homes along McRoberts Avenue between Kitchener Avenue and Summit Avenue.

4. Install storm sewers along Chambers Avenue, Rosethorn Avenue and Silverthorn Avenue between Rogers Road and the south city limits. The storm sewers would be connected to the north trunk sewer along Rogers Road.

5. Disconnect the front yard roof downspouts for the homes bounded by Glenholme Avenue, Bansley Avenue and Vaughan Road. Alternatively, roof downspout flow restrictors could be installed in all downspouts within the same area.

6. The 900 mm diameter combined sewer along Atlas Avenue (segment A309) should be connected to the Rogers Road trunk sewer (segment A60). This will reduce the high surcharge levels which result in basement flooding south of Rogers Road on Atlas Avenue.

The total volume of detention tank storage required (excluding the Caledonia Park storage facility) is 16,695 m^3. This works out to be an average of 145 m^3 per ha of controlled lands. The average release rate from each tank is 40 l/s . The locations of the detention tanks were based on a field reconnaisance and are preliminary in nature. The required volume of each tank is given in Table 2-8. It should be noted that at the detail design stage, it may be possible to reduce the volume requirements by storing water on the roadway surfaces adjacent to the tanks. Storage of water on roadway surfaces was not considered in this study due to the diverse nature of the topography.

TABLE 2-8

10 YEAR DETENTION STORAGE VOLUMES - HILLARY AVENUE SYSTEM

Detention Tank Location	Storage Volume (m^3)
1	725
2	725
3	725
4	363
5	363
6	350
7	350
8	846
9	846
10	846
11	846
12	846
13	290
14	218
15	218
16	580
17	435
18	145
19	580
20	218
21	218

Detention Tank Location	Storage Volume (m^3)
22	327
23	327
24	327
25	327
26	435
27	580
28	145
29	145
30	435
31	580
32	290
33	167
34	290
35	290
36	290
37	175
38	268
39	282
40	282
TOTAL	16,695

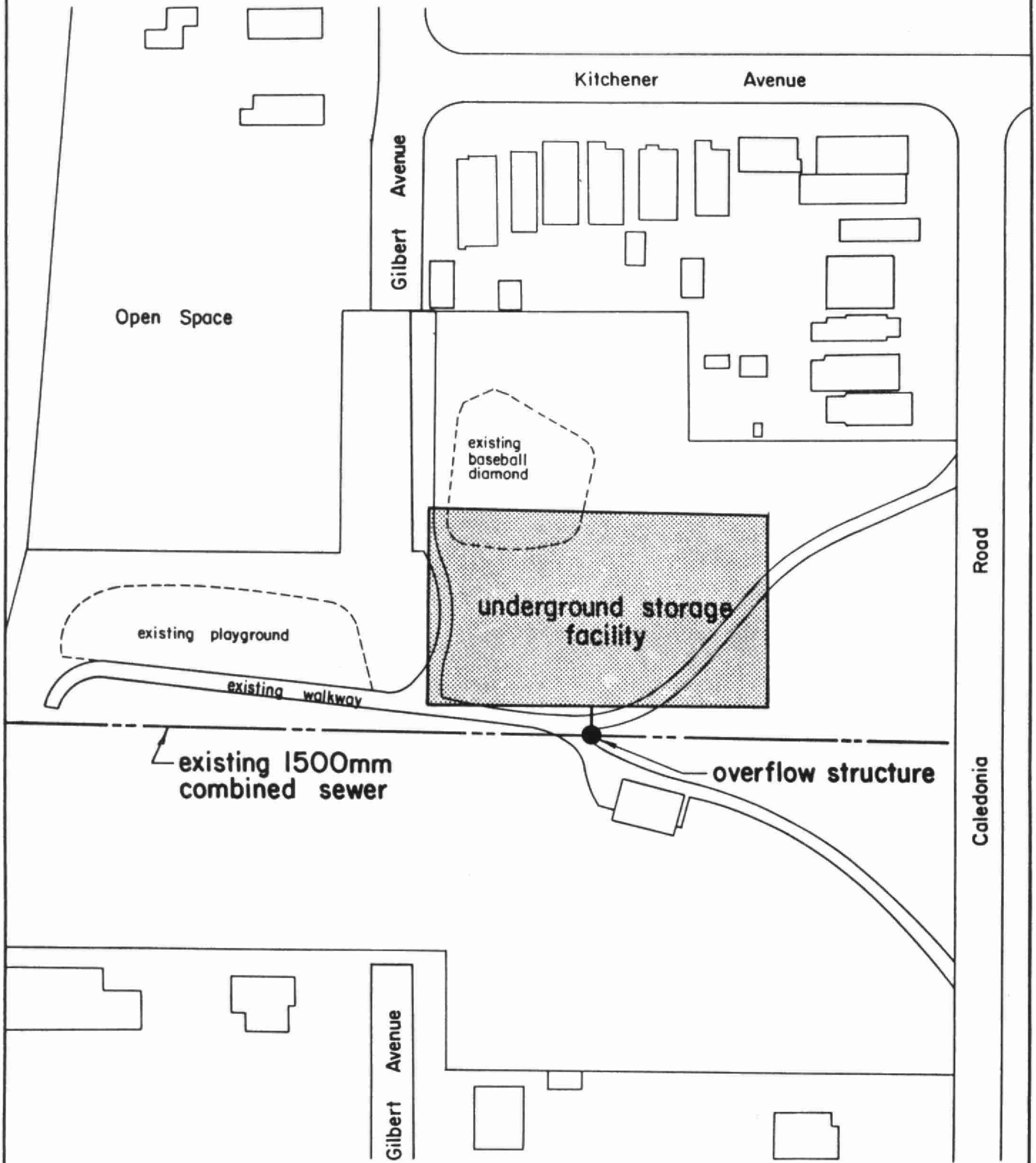
Note: For Detention Tank Locations refer to Plate 2-5

The underground storage facility in Caledonia Park will reduce the surcharge levels in the trunk sewer along Chudleigh Road, Kane Avenue and Dunraven Drive. This, in turn, will eliminate the need for remedial works downstream of the park along Dunraven Drive and Kane Avenue, and will reduce the amount of remedial works in the Chudleigh Road - McRoberts Avenue area. The storage facility would be located in the eastern section of the park (see Figure 2-14). The approximate dimensions of the tank are 60 m x 40 m x 1.8 m deep. The tank storage facility would have roughly 4 m of cover. As the facility, on average, would be in use only one time per year, siltation within the facility should not be a major problem. Access chambers would, however, be constructed to allow manual cleaning as necessary.

Due to the low lying land along McRoberts Avenue, it will still be necessary to disconnect the house connections from the low lying homes along McRoberts Avenue. The house connections will be reconnected to underground storage tanks which will store the domestic sewage, weeping tile drainage and roof water until the storm flow subside. The underground storage tank size as given in Table 2-8 (175 m^3) assumes that either the front yard roof downspouts are disconnected for the low lying homes or roof downspout flow restrictors are installed in all downspouts. If either of the above measures are not carried out, then the required tank volume would increase to 355 m^3.

The existing combined sewers along Chambers Avenue, Rosethorn Avenue and Silverthorn Avenue flow in a northerly direction towards Rogers Road while the streets are graded from north to south. The installation of a detention tank at the southerly limit still resulted in basement flooding at the upper limits of the sewers. For this reason, the recommended remedial works involve constructing storm sewers to convey runoff from the paved and grassed areas to the Rogers Road trunk sewer.

For the area bounded by Glenholme Avenue, Bansley Avenue and Vaughan Road, it is necessary to disconnect the roof downspouts or install roof downspout



CALEDONIA PARK STORAGE FACILITY

figure 2-14

flow restrictors as the combined sewers in this area are very undersized and cannot convey the flows from the roofs, detention tanks and inflows from the restricted catch basins without causing basement flooding.

The feasibility of reducing the detention tank volumes by disconnecting roof downspouts from the combined sewer system and redirecting the roof flows to grassed or paved areas was considered. The results indicated that this practice would not reduce the required detention tank volumes for any of the 2, 5 or 10 year design storms. For all the above design storms the grassed areas are saturated. Furthermore, due to the relatively low release rate from the detention tank, the maximum storage requirements occur well after (30 minutes for the design storm used in this study) the peak runoff rates from the grass and paved areas have reached the detention tank or combined sewer. Delaying the runoff from the roof areas by discharging the runoff to grassed areas will therefore not reduce the storage requirements of the detention tanks.

5.2 Recommended Remedial Works for Combined Sewer Overflows

After the remedial works required to eliminate basement flooding for the 10 year design storm were selected, it was then determined what additional remedial works would be required to eliminate combined sewer overflows within the Hillary Avenue sewer system for each of the five storm events in 1979.

Several alternatives which would eliminate combined sewer overflows were considered. The practice of providing additional inlet control and detention tank storage in areas which were not subject to basement flooding was looked at. However, it became apparent that the cost to install numerous small tanks (including the associated road restoration costs) in order to eliminate combined sewer overflows would be relatively high. Ultimately the combined sewer system would be overcontrolled to convey only the flows that the Hyde Avenue tank and Site 3 regulator could convey to the Black Creek sanitary trunk sewer.

For this reason the combination of inlet control and detention tank storage for areas subject to basement flooding combined with storage at the downstream limit of the system was considered. The head regulating chamber and detention tank as illustrated in Figure 2-12 would be modified in order to reduce the combined sewer overflow volumes at Site 3 and the Hyde Avenue tank. Runoff would be directed into the detention tank facility where possible. The flap gate would be removed as would the overflow from the regulating chamber to the detention facility. The modified arrangement would result in the head within the chamber increasing more slowly, thereby reducing the outlet rate from the tank for the less intense storms. The invert of the tank should, however, be lined with a low flow concrete channel to reduce potential maintenance problems. The modification to the detention facility will require an additional 11 m^3 of storage per ha of controlled area if the 10 year design storm flows are to be stored in the tank without any surface ponding.

With the 10 year remedial works for eliminating basement flooding in place, and using the modified detention tank facilities the five events selected for 1979 were rerun to establish the combined sewer overflow volumes at Hyde Avenue and Site 3. The overflow volumes at the two regulators are given in Table 2-9. The total overflow volumes without any remedial works in place are also given for each storm event. On average, the remedial works will reduce the overflow volumes by 17.2 percent.

Table 2-9 indicates the combined storage volume needed to eliminate combined sewer overflows ranges from 0 m^3 for the May 3, 1979 event to 70,000 m^3 for the July 11, 1979 event. The location and design of the storage facilities required to store the combined sewer overflows would for the first four events be analogous to the facilities discussed in Task 1. In order to store the volumes required for the July 11, 1979 storm, the Hyde Avenue facility would have to be extended northerly to include the existing soccer field. This would likely necessitate that the facility be an underground storage tank.

TABLE 2-9

COMBINED SEWER OVERFLOW VOLUMES WITH
10 YEAR REMEDIAL WORKS FOR BASEMENT FLOODING IN PLACE - HILLARY CATCHMENT

Storm Date	Overflow Volume Hyde Ave	Overflow Volume Site 3	Total Overflow Volume	Total Overflow Volume No Remedial Works
May 3, 1979	0	0	0	0
August 2, 1979	2,400	1,800	4,200	6,100
June 10, 1979	9,700	3,000	12,700	14,700
Sept. 14, 1979	17,300	1,000	18,300	20,600
July 11, 1979	59,600*	10,400	70,000	79,100

Note: * Storage in Caledonia Park facility reduced this number by 2,000 m^3 to 59,600 m^3

All volumes given in cubic metres.

5.3 Remedial Works for Basement Flooding - 5 Year Storm

The recommended solution for providing a 5 year level of protection against basement flooding within the Hillary Avenue sewer system is as follows:

1. Provide inlet control and detention tank storage for the areas as shown in Plate 2-6. A total of 93 ha would be controlled.
2. Construct a 2,500 m^3 underground storage facility in Caledonia Park.
3. Disconnect the house connections from about 30 homes along McRoberts Avenue between Kitchener Avenue and Summit Avenue.
4. Install storm sewers along Chambers Avenue, Rosethorn Avenue and Silverthorn Avenue between Rogers Road and the south city limits. The storm sewers would be connected to the north trunk sewer along Rogers Road.
5. The 900 mm diameter combined sewer along Atlas Avenue (segment A309) should be connected to the Rogers Road trunk sewer (segment A60). This will reduce the high surcharge levels which result in basement flooding south of Rogers Road on Atlas Avenue.

The total volume of storage required (excluding the Caledonia Park storage facility) is 9,100 m^3. This works out to be an average of 98 m^3 per ha of controlled land. The average release rate from the tank is 60 l/s. The required volume of each tank is given in Table 2-10.

5.4 Remedial Works for Basement Flooding - 2 Year Storm

To provide a 2 year level of protection against basement flooding, the following works are recommended:

1. Provide inlet control and detention tank storage for the areas as shown in Plate 2-7. A total of 22 ha would be controlled.

TABLE 2-10

5 YEAR DETENTION STORAGE VOLUMES - HILLARY AVENUE SYSTEM

Detention Tank Location	Storage Volume (m^3)
1	588
2	588
3	196
4	196
5	245
6	245
7	568
8	568
9	568
10	568
11	568
12	196
13	147
14	147
15	294
16	392

Detention Tank Location	Storage Volume (m^3)
22	147
23	147
24	221
25	221
26	221
27	221
28	294
29	196
30	116
31	294
32	147
33	147
34	90
35	185
36	190
37	190
TOTAL	9,100

Note: For Detention Tank Locations refer to Plate 2-6

2. Disconnect the house connections from about 20 homes along McRoberts Avenue between Kitchener Avenue and Summit Avenue.

The total volume of storage required is 1,200 m^3. This works out to be an average of 54 m^3 per ha of controlled land. The average release rate from each tank is 60 l/s. The required volume of each tank is given in Table 2-11.

5.5 Remedial Works - Mt. Dennis and Rockcliffe Catchment Areas

The following methodology was used to estimate the remedial works that would be required for other areas within the City of York (i.e. primarily the Mt. Dennis and Rockcliffe catchment areas), which are bounded roughly by the Humber River to the west, Black Creek to the east, the City limits to the south and Lippincott Street to the north (see Plate 3.1).

The flooding records for the September 14, 1982 storm were inspected. The peak one hour rainfall for this storm was approximately equal to the volume of rainfall which could be expected from an event with a 5 year return frequency and will therefore provide a reasonable indication as to the extent of basement flooding that could be expected for a 5 year storm event. Plan and profiles for the areas that were subject to basement flooding were reviewed and approximate areas where inlet control and detention tanks would be required were established. Using this method it was then estimated that 48 ha of land would require inlet control to eliminate basement flooding for the 5 year design storm. Furthermore, the detention volume required would be 48 ha x 98 m^3/ha or approximately 4,700 m^3.

The ratio of the 10 year detention tank volume to the 5 year volume for the Hillary Avenue sewer system which is equal to 1.7 including the Caledonia Park underground storage) was used to estimate the 10 year detention volume requirements. The same procedure was used to establish the 2 year detention tank storage requirements. For the 2,5 and 10 year design storms it was therefore estimated that 600 m^3, 4,700 m3 and 8,400 m^3 of detention

TABLE 2-11

2 YEAR DETENTION STORAGE VOLUMES

HILLARY AVENUE SYSTEM

Detention Tank Location	Storage Volume (m^3)
1	162
2	108
3	135
4	135
5	81
6	81
7	162
8	162
9	50
10	54
11	92
TOTAL	1,200

tank storage would be required to eliminate basement flooding for the Mt. Dennis and Rockcliffe catchment areas.

To estimate the storage required to eliminate combined sewer overflows data provided by the MOE was utilized. The data provided listed the percentage of overflow volume which occurred at the Rockcliffe, Mt. Dennis and Hillary Avenue (Site 3 and Hyde Avenue) regulators for each of the five storm events. The average of the five events was then used in conjunction with the combined sewer overflow volumes at the Hillary Avenue sites in order to estimate the storage volume required at Mt. Dennis and Rockcliffe. The average percentage overflows are; Hillary Avenue regulators 53.6; Mt. Dennis 33.4; Rockcliffe 13.0.

Table 2-12 lists, for each combined sewer overflow event, the estimated required storage volumes at each of the Rockcliffe, Mt. Dennis and Hillary Avenue regulators. For the storage volumes given, it is assumed that the 10 year remedial works for eliminating basement flooding are in place.

Two other areas within the City of York experience basement flooding. The Weston area which is bounded by the Humber River, north City limits, Lawrence Avenue and Jane Street is serviced by a separate sewer system and was therefore not considered in this study. The second area (refered to as the Westmount-Pinewood area) is located between Dufferin Street, Earlsdale Avenue, Humewood Gardens, Bathurst Street and the south City limits. This area is serviced by a combined sewer system which drains southerly to the City of Toronto. A detailed study has been completed for a portion of this area (Reference 9). The costs as presented in this report were used in conjunction with the available flooding records to estimate 2, 5 and 10 year costs for the inlet control works (see Section 6) within the Westmount-Pinewood area.

5.6 Roof Downspout Disconnection for Combined Sewer Overflow Events

Although not in the Terms of Reference, a sample run was carried out to

TABLE 2-12

COMBINED SEWER OVERFLOW STORAGE VOLUMES

Storm Date	Overflow Volume Hyde Ave	Overflow Volume Site 3	Overflow Volume Mt. Dennis	Overflow Volume Rock-cliffe	Total Overflow Volume
May 3, 1979	0	0	0	0	0
August 2, 1979	2,400	1,800	2,600	1,000	7,800
June 10, 1979	9,700	3,000	7,900	3,100	23,700
Sept. 14, 1979	17,300	1,000	11,400	4,400	34,100
July 11, 1979	59,600	10,400	43,600	17,000	130,600

Note: All volumes given in cubic metres

estimate, for the less intense storms, the impact of disconnecting the front yard downspouts on the combined sewer overflow volumes. Discharging the runoff from the roof areas to pervious (grass) areas may reduce combined sewer overflows as a larger percentage of the rainfall would, for the less intense storms, infiltrate to the soil.

Only front yard downspouts were assumed to be disconnected as poor lot drainage and narrow lots would render rear yard disconnection to be unfeasible.

The September 14, 1979 storm was selected for analysis. In order to determine the direct impact of disconnecting the downspouts, no additional remedial works were considered to be in place. The results indicated that front yard roof downspout disconnection of all homes in the Hillary Avenue catchment area would reduce the combined sewer overflow volume at the Hillary Avenue regulators from 20,600 m^3 to 14,900 m^3 or 28 percent. The total roof area which was disconnected equalled 78 ha, of which 38 ha represents the net reduction in impervious area (i.e. 50 percent of the front yard downspouts discharged to grassed areas).

6 - ESTIMATED COSTS OF ALTERNATIVE REMEDIAL WORKS

Cost estimates for the remedial works as described in Chapter 5 were prepared, and are presented in Table 2-13 through Table 2-17 and Figure 2-15. The estimates were based on 1984 construction costs and include installation, restoration, operation and maintenance costs, land costs and a 20 percent engineering and contingency allowance. All annual costs were converted to capital costs using a 70 year time period and a net interest rate (interest rate minus inflation rate) of 7 percent.

In considering costs for the detention tank and inlet control works,several typical sites were selected and estimates were prepared. Allowance has been made for vertical sheeting in open cut trenches, sawcuts of the existing asphalt, disposal of all excavated material from roadways, watermain and water service relocations. Restoration will include a minimum of 450 mm granular "A" and 100 mm hot mix asphalt for a minimum width of one driving lane (3.5 m). The cost estimates for the catch basin sealing assume that a concrete cap is installed and the adjacent curb is replaced (see section 5). The unit costs for various components are listed in Table 2-13.

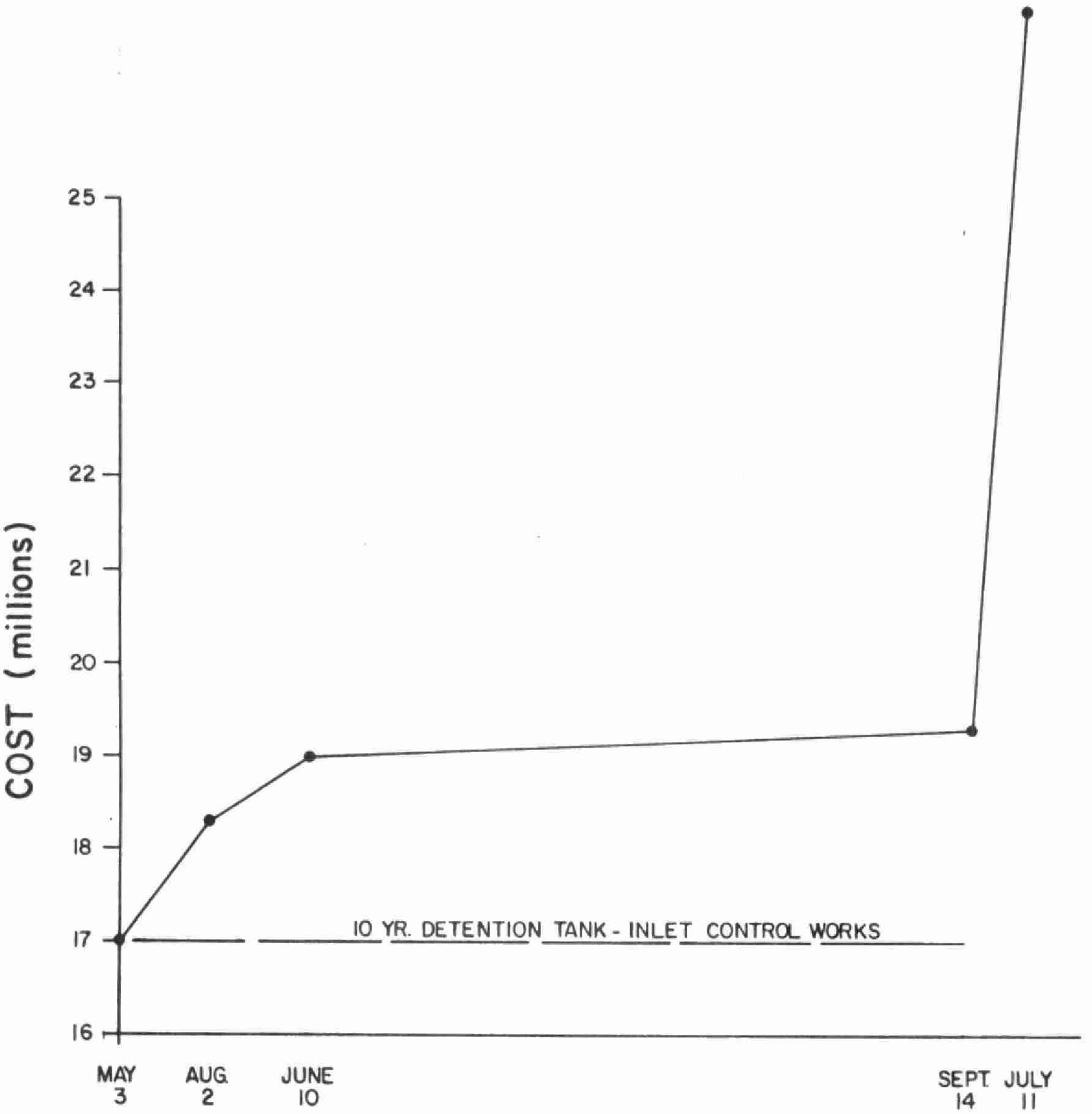
Operation and maintenance costs for the detention tanks within the City of York were estimated based on discussions with City of York staff. Allowance has been made for a four man crew and associated equipment to clean each facility once per year. The estimated costs, assuming a 2, 5 or 10 year level of protection are $14,000, $30,000 and $30,000 respectively. In present value terms this works out to be $200,000 for a 2 year level of protection, or $420,000 for a 5 or 10 year level.

The cost given for the underground storage facility in Caledonia Park was prepared in a similar fashion to the costs prepared for Task 1 and includes excavation in stable till type soils, construction of cast-in-place tank, piping, access manholes, backfill with native material, disposal of surplus excavation and restoration with topsoil and sod.

TABLE 2-13

UNIT COST FOR DETENTION TANKS AND INLET CONTROL WORKS

Component	Unit Cost
Detention Tank Installation	$\$520/m^3$
Storm Sewer Installation	$630/m
Underground Storage Facility Caledonia Park	$\$134/m^3$
Catchbasin Restrictors	$ 75/CB
Catch Basin Sealing	$350/CB
Roof Downspout Disconnection	$100/downspout



ESTIMATED TOTAL COSTS FOR - 10 YEAR LEVEL OF FLOOD PROTECTION FOR THE HILLARY, MT. DENNIS, ROCKCLIFFE AND WESTMOUNT-PINEWOOD CATCHMENTS AND ELIMINATING COMBINED SEWER OVERFLOWS AT THE FOUR REGULATORS

figure 2-15

The cost estimates for the storage facilities at the four Black Creek regulators were, except for the July 11, 1979 estimates, based on the estimates given in Task 1. The storage requirements for the July 11, 1979 event were larger than the volumes considered in Task 1 and a separate estimate was therefore prepared. The cost estimates for the storage facilities assume that the 10 year inlet control works are in place.

Table 2-14 lists the cost estimates for the Hillary Avenue system for all works excluding the storage facilities at the Hyde Avenue tank and Site 3 regulator. Cost estimates are given for the 2, 5 and 10 year design storms. As the storage facility within Caledonia Park is located within City owned lands, no land cost was allowed for.

In preparing the cost estimates for the detention tank and inlet control works for the Mt. Dennis and Rockcliffe catchments, the cost estimates for the Hillary Avenue System and the drainage areas to be provided with inlet control (see section 5.5) were used. In addition, a pro-rata cost (based on drainage areas) for an underground storage facility similar to the facility proposed for Caledonia Park was included for the Mt. Dennis and Rockcliffe catchments. The cost estimates for the detention tank and inlet control works for Mt. Dennis and Rockcliffe catchments are given in Table 2-15.

The cost estimates for the Westmount-Pinewood Area were based on a previous study (see section 5.5). The estimates to provide detention tanks and inlet control works for a 2, 5 and 10 year level of protection are $625,000, $1,300,000 and $1,800,000 respectively.

The cost estimates for the storage facilities to reduce combined sewer overflows are given in Table 2-16 and Table 2-17. The estimates assume three facilities will be used, and will store the overflow from the Hyde Avenue tank, the Mt. Dennis overflow and the Site 3 plus Rockcliffe overflows combined. Estimates for closed (detention tanks) and open (detention basins) facilities are listed. The cost estimates given include allowances for operation, maintenance and replacement.

TABLE 2-14

COST ESTIMATES FOR DETENTION TANKS AND INLET CONTROL WORKS
HILLARY AVENUE CATCHMENT

Component	Design Storm		
	2 Year	5 Year	10 Year
Detention Tanks	620,000	4,740,000	8,850,000
Storm Sewers	0	270,000	270,000
Caledonia Park Storage	0	340,000	530,000
Catch Basin Restrictors	0	10,000	10,000
Catch Basin Sealing	20,000	80,000	100,000
Roof Downspout Disconnection	0	0	10,000
CAPITAL COST	640,000	5,440,000	9,770,000
Present Value Cost for Operation and Maintenance	85,000	225,000	225,000
TOTAL COST	$725,000	$5,665,000	$9,995,000

TABLE 2-15

COST ETIMATES FOR DETENTION TANK AND INLET CONTROL WORKS
FOR THE MT. DENNIS AND ROCKCLIFFE SYSTEM

Component	Design Storm		
	2 Year	5 Year	10 Year
Detention Tanks	340,000	2,600,000	4,680,000
Underground Park Storage	0	200,000	290,000
Inlet Control Works	10,000	50,000	60,000
CAPITAL COST	350,000	2,850,000	5,030,000
Present Value Cost for Operation and Maintenance	65,000	140,000	140,000
TOTAL COST	$415,000	$2,990,000	$5,170,000

TABLE 2-16

TOTAL COST ESTIMATES FOR PROVIDING DETENTION TANKS FOR EACH REGULATOR

Storm Date	Location			
	Hyde Ave	Site 3 and Rockcliffe	Mt. Dennis	Total
May 3, 1979	0	0	0	0
August 2, 1979	520,000	590,000	570,000	1,680,000
June 10, 1979	1,120,000	810,000	830,000	2,760,000
Sept. 14, 1979	1,760,000	760,000	1,040,000	3,560,000
July 11, 1979	4,170,000	2,610,000	3,310,000	10,090,000

TABLE 2-17

TOTAL COST ESTIMATES FOR PROVIDING DETENTION BASINS FOR EACH REGULATOR

Storm Date	Location			
	Hyde Ave	Site 3 and Rockcliffe	Mt. Dennis	Total
May 3, 1979	0	0	0	0
August 2, 1979	280,000	290,000	380,000	950,000
June 10, 1979	360,000	440,000	490,000	1,290,000
Sept. 14, 1979	540,000	410,000	520,000	1,470,000
July 11, 1979	-	-	-	-

Note: Refer to Table 2-12 for Storage Volumes.

One final set of estimates is given (Figure 2-15). This figure combines the estimates for providing a 10 year level of flood protection for the Hillary, Mt. Dennis, Rockcliffe and Westmount-Pinewood catchments and reducing combined sewer overflows at the four regulators. The five storm events in 1979 which were analysed have been used as a basis for developing the curve. In establishing this curve, it was assumed that the Rockcliffe, Mt. Dennis and Site 3 facilities would be combined. Furthermore, this facility would be a closed tank. For the Hyde Avenue tank it was assumed that an above ground facility would be constructed to accommodate all but the July 11, 1979 event. For this event, the large storage volume necessitates that a closed tank be used, due to space limitations.

7 - CONCLUSIONS

1. For this task the largest catchment (Hillary Avenue) was analyzed in detail. The findings for this study area were then used to estimate the required remedial works and estimated costs for three smaller catchments (Mt. Dennnis, Rockcliffe and Westmount-Pinewood). The Weston Area is served by a separate sewer system and was therefore not considered in this study.

2. Results for the Hillary Avenue catchment area indicated that basement flooding will occur for storms equalling or exceeding the 2 year storm. For the 10 year design storm it was found that about 25 percent of the area would be subject to basement flooding. The flooding problems are due in part to undersized laterals, isolated low lying areas and for the 5 and 10 year design storms, to undersized combined trunk sewers. Inspection of the flooding records suggest that flooding problems within the Rockcliffe and Mt. Dennis catchments would be slightly less severe.

3. A preliminary analysis was carried out in order to approximate both the volume of combined sewer overflows and number of combined sewer overflow events which would occur at the Hyde Avenue and Site 3 regulator. Analysis of five storm events during the April to October 1979 period suggested that under existing (1985) conditions combined sewer overflows could be expected to occur roughly ten times per year. The total volume of combined sewer overflows at the two regulators (Site 3 and Hyde Avenue) ranges from 0 m^3 for an event which would, on average, be exceeded 10 times per year to 79,100 m^3 for an event which has approximately a 3 year return period. If the Mt. Dennis and Rockcliffe regulators are included, the total combined sewer overflow volumes for the above two events would be 0 m^3 and 147,500 m^3 respectively.

4. Remedial works which have been constructed or are scheduled to be constructed during 1985 have reduced both combined sewer overflow volumes to the Black Creek and basement flooding problems. At the end of 1985 approximately 25 percent of the Hillary Avenue catchment will be separated and 4 percent will have inlet control and detention tank facilities. Separated in the context of this study means that grassed and paved areas drain to the storm sewers and therefore do not contribute runoff to the Hillary Avenue combined trunk sewer system.

5. The most cost effective method for reducing basement flooding and reducing the frequency and magnitude of combined sewer overflows is to construct detention tanks and inlet control works in areas which are subject to basement flooding and storage facilities to reduce combined sewer overflows adjacent to the Hyde Avenue and Site 3 regulators. Providing only detention tanks and inlet control works would reduce combined sewer overflow volumes by 17 percent on average.

6. Source control (i.e. disconnecting the roof downspouts from the combined sewer and discharging the runoff to grassed or paved areas) will not reduce the required detention tank volumes for any of the 2, 5 and 10 year design storms.

7. Source control will help reduce combined sewer overflows for the less intense storms. A sample run indicated that the volume of combined sewer overflow for the Hillary Avenue system could be reduced by 28 percent if all front yard downspouts within the the study area were disconnected.

8. The feasibility of providing local storm sewers was considered, however due to the number of trunk sewers which are surcharged for the 5, and 10 year design storms and the asssociated cost of installing storm sewers (see Task 3), the recommended remedial works include the construction of only three local storm sewers.

9. Ponding of water on the roadways adjacent to the detention tanks was not considered in this study due to the diverse nature of the topography within the study area. Previous studies (Reference 12, 13) suggest that this method and other methods of inlet control may reduce the required detention tank volumes and ultimately the total cost to eliminate basement flooding. Appendix 2-1 outlines the costs for eliminating basement flooding problems for other areas within Canada and the United States, utilizing the inlet control method.

10. The capital cost and total cost (capital cost plus converted annual cost as shown in brackets) to provide a 10 year level of protection against basement flooding and reduce combined sewer overflow to an average of one event per year is $18,840,000 ($19,350,000), of which $16,600,000 ($17,020,000) is required for the detention tank and inlet control works, and $2,240,000 ($2,330,000) for the storage facilities adjacent to the existing Black Creek regulators. Other costs for various levels of combined sewer overflow and basement flooding elimination are given in Section 6.

LIST OF ABBREVIATIONS

The following is a list of abbreviations which have been used in the text.

ft	=	feet
ft^2	=	square feet
ft^3	=	cubic feet
Igpd/cap	=	Imperial gallons per day per capita
Igpm	=	Imperial gallons per minute
in	=	inches
l/s	=	litres per second
l/s/cap	=	litres per second per capita
m	=	metres
m^2	=	square metres
m^3	=	cubic metres
m^3/sec	=	cubic metres per second
mm	=	millimetres

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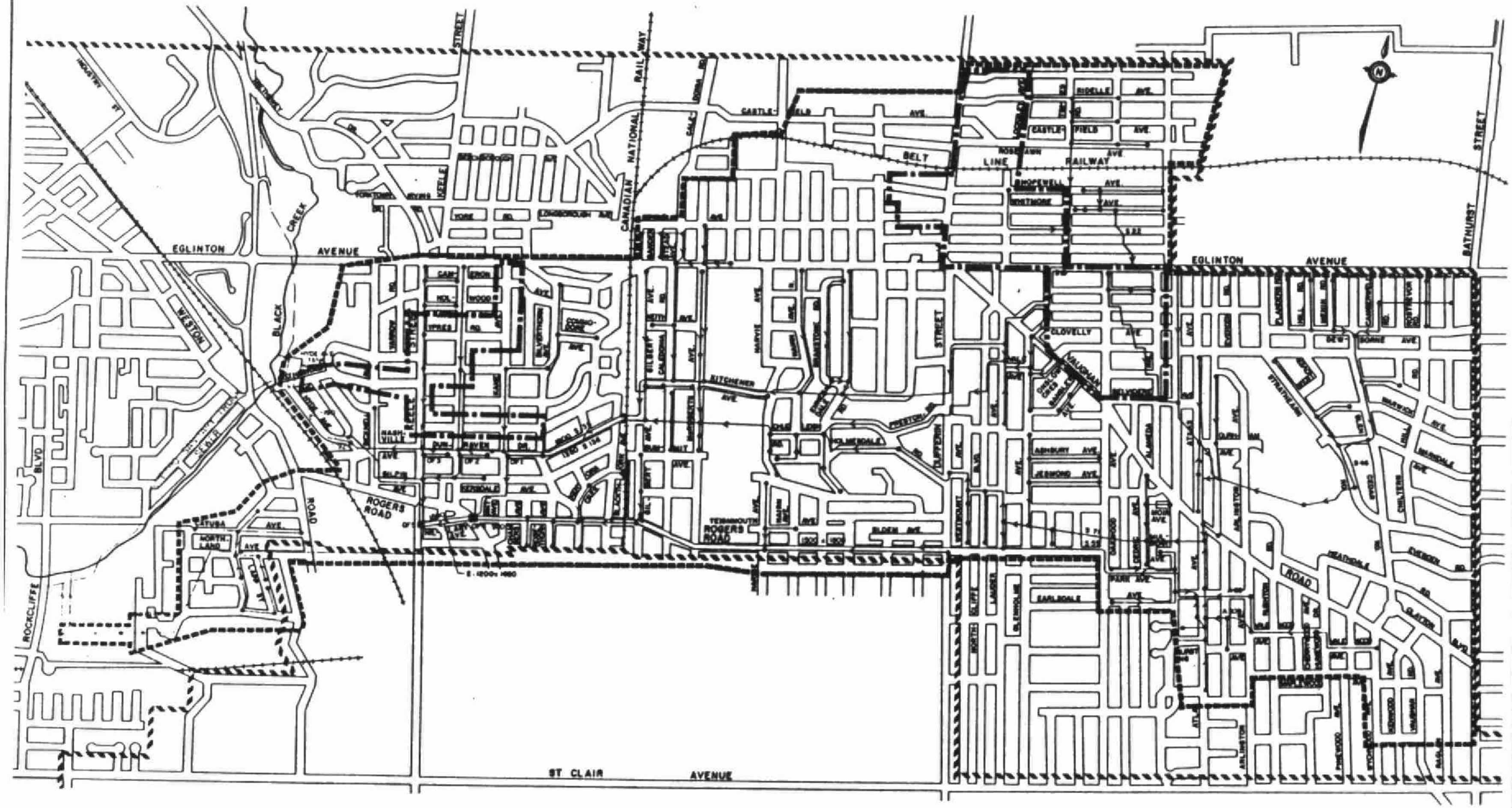
APPENDIX 2-1

COST SUMMARY FOR FLOOD RELIEF PROJECTS USING INLET CONTROL

Location	Year	Area served (ha)	Construction Cost $	Cost Hectare
Sarnia *	1984	181	350,000	1,930
Niagara Falls*	1981	225	685,000	3,040
Mississauga *	1979	100	350,000	3,500
Regina *	1980	891	9,500,000	10,660
City of York (Hillary Ave)	1984	630	9,490,000	15,060
Cleveland *	1980	47	725,000	15,420
Windsor *	1982	89	2,600,000	29,200

* Remedial works have been installed



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

SYSTEM SEGMENTATION Hillary Avenue Catchment

0 100 300 500 700
scale metres

legend

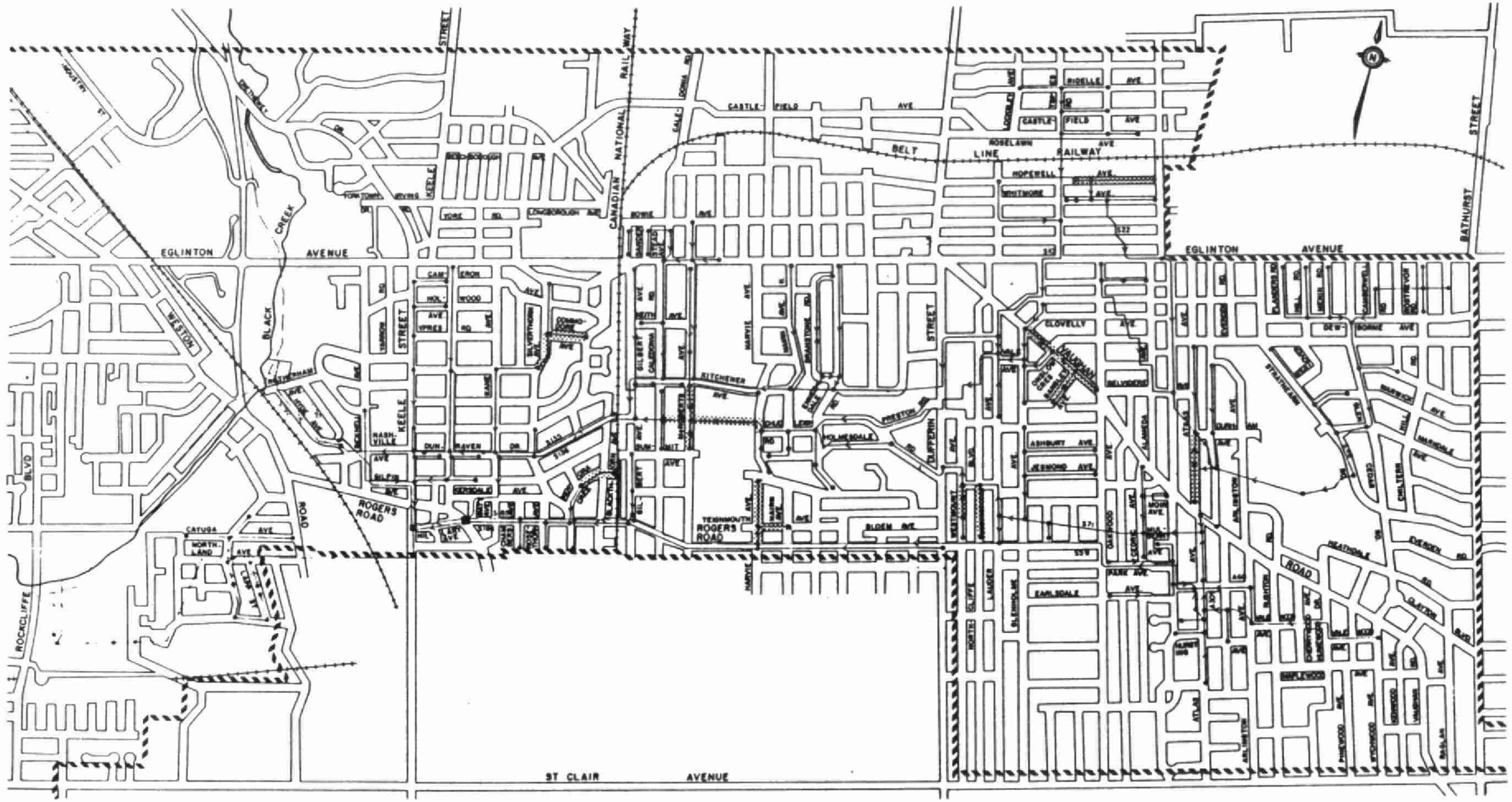
existing combined sewer with flow direction

limit of study area

city limits

approximate limits of sewer separation between [illegible]

paul theil associates limited



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

2 YEAR STORM BACKWATER LEVELS Hillary Avenue System

0 100 300 500 700

scale metres

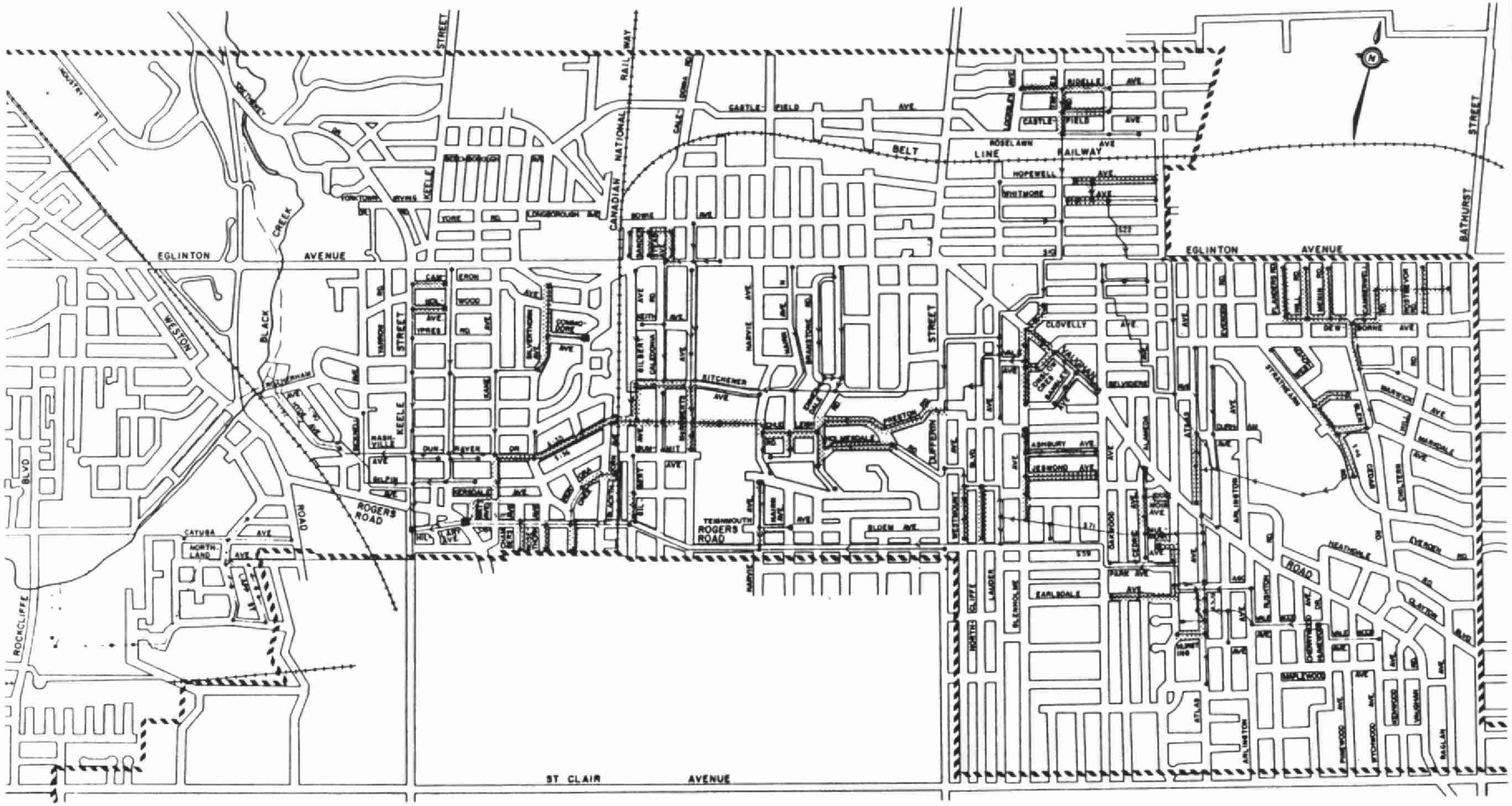
legend

paul theil associates limited

Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

5 YEAR STORM BACKWATER LEVELS
Hillary Avenue System



0 100 300 500 700
scale metres

legend

existing combined sewer with flow direction

water levels exceeding road surface elevation

water levels between basement floor elevation and road surface elevation

water levels below basement floor elevation

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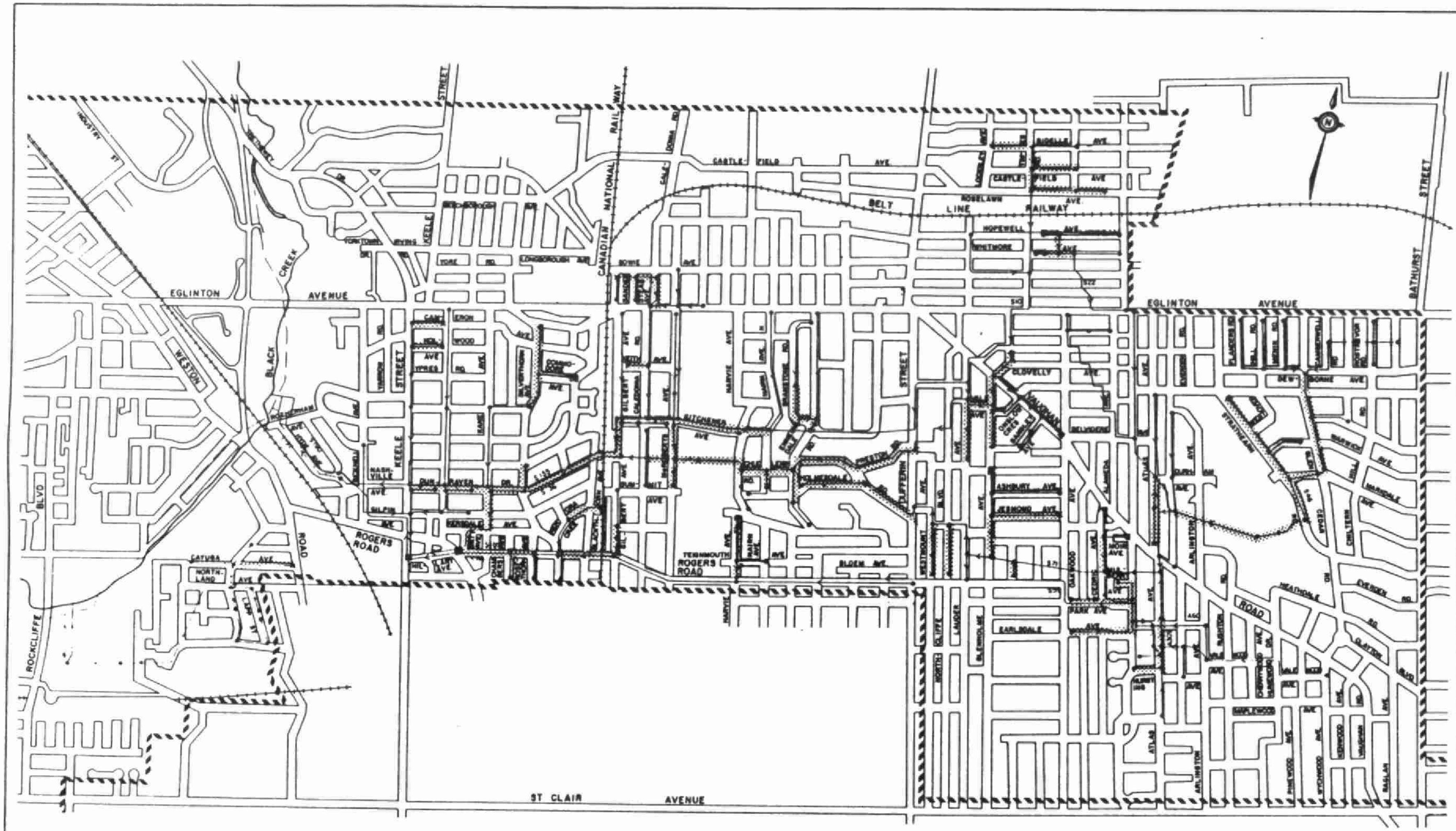


paul theil associates limited

Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

10 YEAR STORM BACKWATER LEVELS Hillary Avenue System

0 100 300 500 700

scale metres

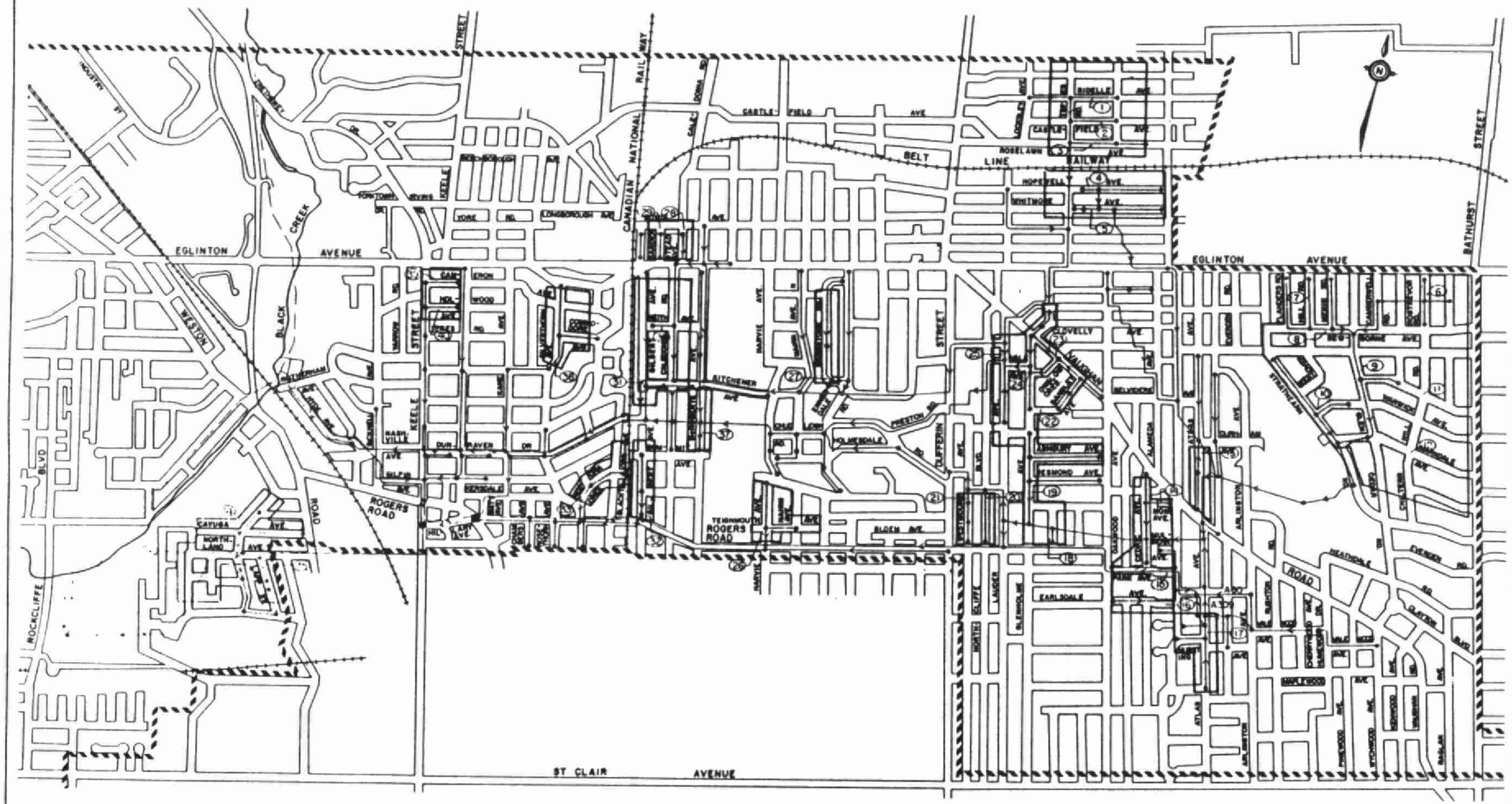
legend

existing combined sewer with flow direction

water levels exceeding surface elevation

water levels between flow elevation and surface elevation

water levels below flow elevation



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

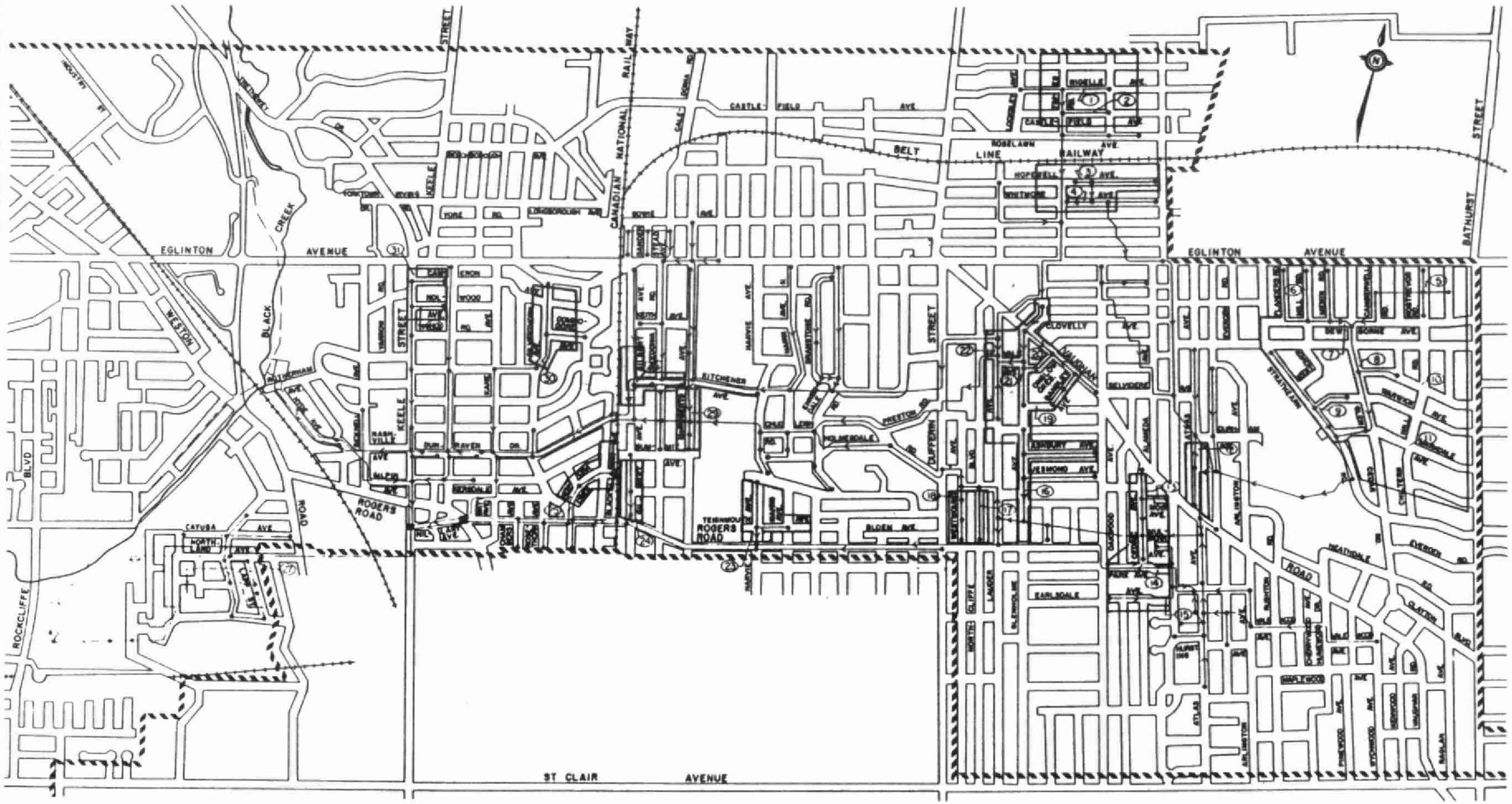
REMEDIAL WORKS FOR BASEMENT FLOODING
10 Year Storm

0 100 300 500 700

scale metres

legend

paul theil associates limited



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

REMEDIAL WORKS FOR BASEMENT FLOODING 5 Year Storm

0 100 300 500 700

scale metres

legend

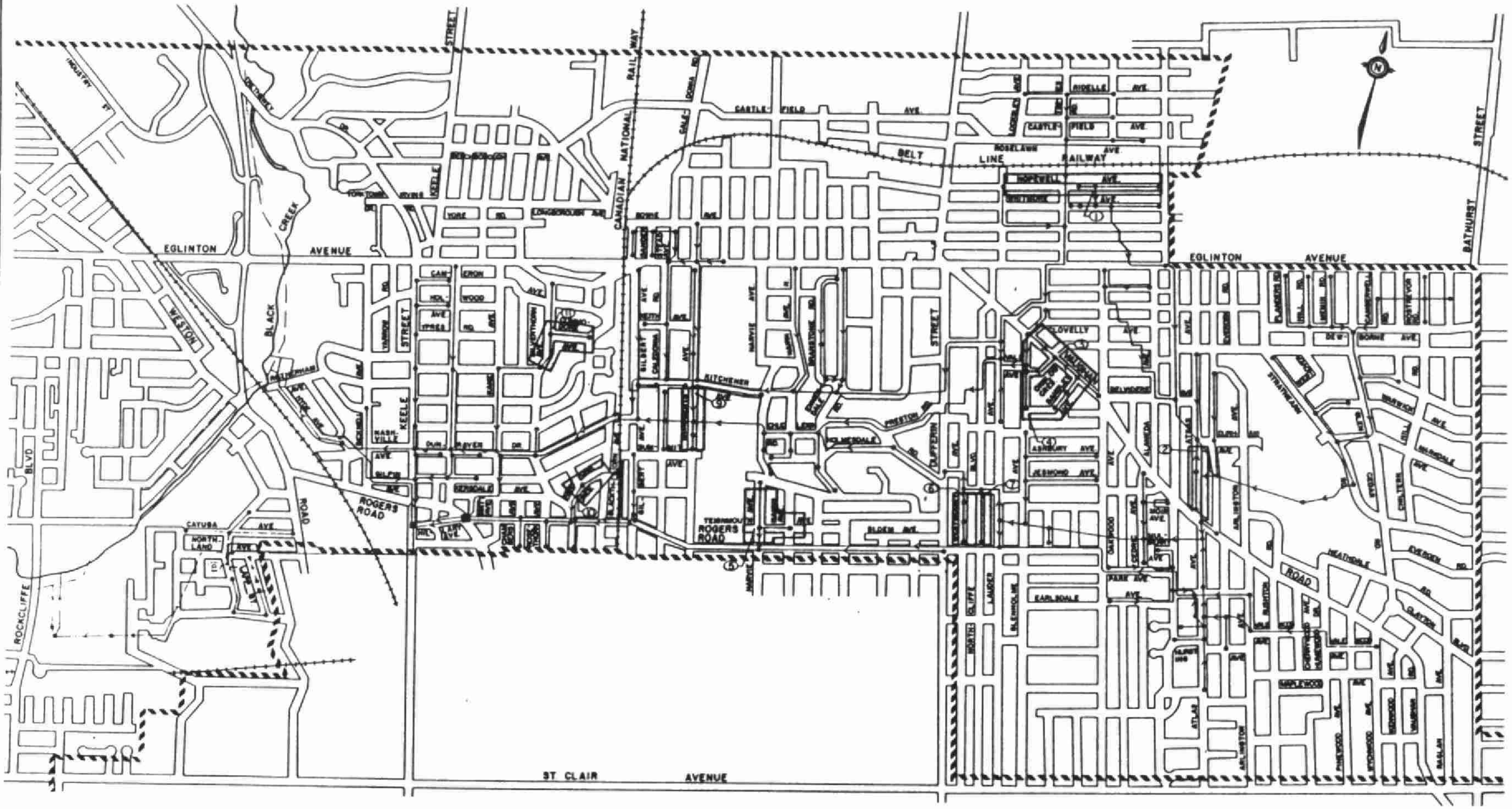
existing combined sewer with flow direction

detention tank locations

areas with inlet control

paul theil associates limited

plate 2



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

REMEDIAL WORKS FOR BASEMENT FLOODING 2 Year Storm

legend

existing combined sewer with flow direction

detention tank locations

areas with inlet control

paul theil associates limited

plate

SECTION 3

TASK 3

Separation of Combined Sewers

TASK 3

SEPARATION OF COMBINED SEWERS

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FEASIBILITY STUDY AND COSTING OF
PROPOSED POLLUTION CONTROL MEASURES
IN THE HUMBER SEWERSHED

TASK 3

SEPARATION OF COMBINED SEWERS

1 - INTRODUCTION

1.1 General

During the last several years, the City of York has been implementing a scheme which will separate the original combined sewer system. To date considerable storm trunk sewer construction has been carried out in various areas of the City. However, local storm sewers which make up the greatest portion of the scheme have only been installed to a limited extent. Therefore, combined sewers which are incapable of handling the runoff from the more severe storm events still serve much of the City. As a result, basement flooding is widespread and combined sewer overflows to the Black Creek occur frequently.

1.2 Objective

The main objective of Task 3 is to estimate the cost of sewer separation within the City, based on late 1984 prices. The costs to complete the scheme in each catchment will be determined based on information available from previous detailed studies. The results from this Task will enable the management of the TAWMS to compare the merits of this approach with those of other methods which may be used to achieve similar objectives.

Section 2 of this task outlines the estimated costs to construct the remaining trunk and local sewers. Construction of these works will provide

only partial separation of the sewer system, as a significant percentage of the roof downspouts will still be connected to the original combined sewer. Section 3 deals with the estimated costs to provide a fully separate sewer system.

1.3 Study Methodology

In general, our approach to this task consisted of reviewing the previous consultant's reports, establishing the present status of the City's separation program, and updating all costs required to complete the scheme. The initial step of the study was a thorough review of the Gore and Storrie reports of April 1967.

Meetings were then held with the City to determine the trunk and local storm sewer installations that have taken place up to and including 1984 and what construction was scheduled for 1985. The boundaries of the catchment areas were verified based on contour information from the Metropolitan Toronto Area Municipal Map Series and on-site investigtions.

Plan and profile drawings for all trunk sewer locations were obtained from the City of York. The sewers indicated in the Gore and Storrie reports were plotted on the plans, using the sizing, depths and grades stated in the reports. The design was adjusted only where obvious conflicts with existing sewers or service connections were detected, and only where such adjustment was more practical than the original intent. No analysis of the original design or design criteria was undertaken. Cost estimates to complete the implementation of the City's separation scheme were prepared. Every length of trunk storm sewer required to complete the system was estimated in detail. Cost summaries and breakdown by street for each of the western, central and eastern areas are presented in the following sections.

It was beyond the scope of this study to prepare detailed cost estimates for all the lengths of local storm sewers remaining to be installed. Accordingly, for proposed local storm sewer construction, a sample area was estimated in detail. The sample section contained sewer sizes from 300 mm to 600 mm at various depths. This estimate was used to obtain the average cost per metre ($605) which was applied to determine the cost of the remaining local storm sewer construction.

2 - COSTING ANALYSIS

2.1 Existing Conditions

The City of York has carried out considerable storm sewer trunk construction in the three basic catchment areas (western, central and eastern). Local storm sewers have only been installed to a limited extent. The separation program was reviewed with the City and its present status documented. Plates 3-1 and 3-2 show the trunk and local storm sewer installations that have taken place up to and including 1984. Storm sewer construction for 1985 has been included in the cost estimates as a separate item. The costs for these sewers have been taken from the City of York Departmental Ranking of 1985 Capital Projects.

Given below, in sections 2.2, 2.3 and 2.4, are cost estimates for the remaining sewer works as outlined in the Gore & Storrie report. Section 2.5 provides an estimate for works within an additional area in the City of York.

2.2 Western Area

The existing and proposed trunk sewers to service the western area are shown on Plate 3-1. As can be seen from the drawing, very little local sewer construcion has taken place. For 1985, the Scarlett Road outlet (W11) has been scheduled for construction at a cost of $1,600,000. The remaining trunk sewers are estimated to cost $5,760,000, for a total of $7,360,000 as shown in Table 3-1 (see Appendix 3-1 for breakdown of sewer costs for each trunk sewer segment). Local sewers on Lambton and Brownville Avenues are to be constructed in 1985 at a cost of $92,000. Approximately 26,460 metres of local sewers will remain at a cost of about $16,000,000 (see Appendix 3-1 for breakdown of costs for local sewer construction). To allow for engineering and contingencies 20 percent is added, giving a total estimated cost for this area of $28,200,000 as shown in Table 3-1.

TABLE 3-1

WESTERN AREA - COST ESTIMATE SUMMARY

Sewer	1985 Construction Costs	Remaining Costs	Total Costs
Trunks	$1,600,000	$5,760,000	$7,360,000
Locals	$ 92,000	$16,000,000	$16,092,000
TOTAL			$23,452,000
20% Engineering and Contingencies			4,690,400
GRAND TOTAL			28,142,400
		SAY	$28,200,000

2.3 Central Area

The existing and proposed trunk sewers servicing the central area are shown on Plate 3-2. Similar to the western area, local sewer construction has been minimal to date. The amount of $387,000 has been scheduled to complete trunk sewer C4 (Ypres, Kane, Eglinton) in 1985. The remaining trunk sewers are estimated to cost $3,995,000 for a total cost of $4,382,000 as shown in Table 3-2 (see Appendix 3-2 for breakdown of sewer costs for each trunk sewer segment).

Local sewers scheduled for construction in 1985 are listed in Appendix 3-2 and will cost $1,310,000. Aproximately 48,000 metres of local sewers will remain to be constructed at an estimated cost of $29,040,000 (see Appendix 3-2 for breakdown). An addition of 20 percent for engineering and contingencies will require total funding of $41,700,000 to complete the separation in this area as shown in Table 3-2.

2.4 Eastern Area

The existing and proposed trunk sewers serving the eastern area are shown on Plate 3-2. On an area basis, it would appear that local sewer construction has been carried out to a greater extent than in the other areas. However, 15,700 metres of local sewers still remain to be constructed at an estimated cost of $9,498,500. Only $46,000 has been allocated in 1985 for local sewer construction on Vaughan Road from Oakwood to Glenora, as shown in Table 3-3. The City's program does not provide for any trunk sewer installations in this area in 1985, such that the total remaining expenditure for trunk sewers is $2,710,000. The total cost to complete the sewer separation scheme in this area, including 20 percent for engineering and contingencies, is $14,800,000 as shown in Table 3-3 (see Appendix 3-3 for breakdown of estimated sewer costs for this area).

2.5 Weston Area and Westmount-Pinewood Area

Two other areas within the City of York experience basement flooding. The

TABLE 3-2

CENTRAL AREA - COST ESTIMATE SUMMARY

Sewer	1985 Construction Costs	Remaining Costs	Total Costs
Trunks	$ 387,000	$3,995,000	$4,382,000
Locals	$1,310,000	$29,040,000	$30,350,000
TOTAL			$34,732,000
20% Engineering and Contingencies			6,946,400
GRAND TOTAL			41,678,400
		SAY	$41,700,000

TABLE 3-3

EASTERN AREA - COST ESTIMATE SUMMARY

Sewer	1985 Construction Costs	Remaining Costs	Total Costs
Trunks	Nil	$2,710,000	$2,710,000
Locals	$ 46,000	$9,498,500	$9,544,500
TOTAL			$12,254,500
20% Engineering and Contingencies			2,450,900
GRAND TOTAL			14,705,400
		SAY	$14,800,000

Weston Area which is bounded by the Humber River,north City limits, Lawrence Avenue and Jane Street is serviced by a separate sewer system. A separate analysis, beyond the scope of this study, would be required to determine the necessary remedial works to alleviate flooding problems.

The second area (referred to as the Westmount-Pinewood Area) is located between Dufferin Street, Earlsdale Avenue, Humewood Gardens, Bathurst Street and the south City limits. This area is serviced by a combined sewer system which drains southerly to the City of Toronto. Due to a previous agreement between the City of Toronto and the City of York, a storm sewer outlet to convey storm flows from the Westmount-Pinewood area was not provided. The basement flooding problems for this area could be alleviated by providing inlet control and underground detention tanks (Task 2). The estimated cost of these works is $1,800,000.

2.6 Costing Summary

The total estimated construction costs to complete the separation scheme in the City of York (excluding the Weston and the Westmount-Pinewood Areas) is shown in Table 3-4. As can be seen, about $17,250,000 is required to complete the remaining trunk storm sewers, and approximately $67,450,000 is needed to construct the local storm sewers,for a grand total of $84,700,000. It should be stressed that these costs cover the type of sewer separation currently being undertaken by the City of York, namely, only surface drainage drained by street catchbasins is conveyed to the storm sewer system. Storm runoff from roofs via downspout connections which provides about 40 percent of the total storm flow to the combined sewer has not been included in these costs.

TABLE 3-4

CITY OF YORK - SEWER SEPARATION PROGRAM
COST ESTIMATE SUMMARY

Sewer	Total Costs (Construction, Engineering, Contingencies)			
	Western Area	Central Area	Eastern Area	Totals
Trunks	$ 8,800,000	$ 5,200,000	$ 3,250,000	17,250,000
Locals	19,400,000	36,500,000.0	11,550,000	67,450,000
TOTALS	$28,200,000	$41,700,000	$14,800,000	84,700,000

An additional $1,800,000 will be requiired to provide remedial works in the Westmount-Pinewood Area

3 - PRIVATE PROPERTY CONNECTIONS

3.1 General

Storm runoff from roofs via downspout connections, contribute about 40 percent of the total storm flow to the combined sewer. As mentioned above, the cost estimates do not make any allowance for the removal of these flows from the combined sewer. Accordingly, the following alternative methods of eliminating or reducing the roof flows were investigated.

3.2 Storm Water Connections

The provision of individual storm water connections to service each dwelling was analyzed as a method of providing complete separation. A site inspection of several streets in the City of York revealed that only downspouts located at or near the front of the houses could feasibly be connected to a new storm service pipe. To connect downspouts at the rear of units in most cases would be impractical, if not impossible, due to the cost of restoration, and constraints imposed by elevation and accessability. The total estimated cost to provide a storm sewer connection from the new storm sewer to the front of each house is about $6,400 per house. The additional costs to provide complete sewer separation by the installation of storm sewer connections for each area is shown in Table 3-5. Total costs for this service for the entire City would be $127,000,000 which is more than one and one half times the cost of the trunk and local sewers required to separate the system.

3.3 Downspout Disconnection

As was stated earlier, runoff from roofs will generate about 40 percent of the total storm flows. It is evident, based on the City's flooding records, that these flows are sufficient enough to cause basement flooding even in some areas of the City of York where sewer separation is complete. Roof connections are also a factor in the frequency of combined sewer overflows.

TABLE 3-5

CITY OF YORK - STORM SEWER CONNECTIONS

COST SUMMARY

Area	Number of Dwelling Units	Estimated Costs @ $6400/unit
Western	6,910	$44,224,000
Central	9,280	$59,392,000
Eastern	3,630	$23,232,000
TOTAL	19,820	$126,848,000
	SAY	$127,000,000

To estimate the total number of connected roofs, a site investigation of several streets in the City of York was carried out. The results of the sample investigation are shown in Table 3-6. The study revealed that 86 percent of the downspouts were connected to the sewer and that there was an average of 2.3 downspouts per unit. Further, about 63 percent of the downspouts that are connected to the combined sewer are situated at the front of the houses.

It is obvious that the disconnection of downspouts and the redirecting of roof runoff to the surface cannot be accomplished in all instances, due to many factors including negative grades, insufficient swale or receiving area for the discharge, proximity to entrances, window wells, etc. Our investigations indicate that in the majority of cases these difficulties would apply to most downspouts located at the rear of the dwelling units and for these reasons should be considered impractical to disconnect. Conversely, based on our site studies, most, if not all downspouts located at the front of the houses should be able to be disconnected and discharged in an acceptable manner.

The costs to disconnect all front downspouts in the City is outlined in Table 3-7. The total cost of $2,500,000 is based on $100 per downspout. A previous study (Reference 3-1) has documented disconnection costs at $70 to $186 per rainwater leader. However, the areas investigated consisted of a relatively small number of homes (less than 50). In this investigation, where only front downspouts are being considered for removal, $100 per downspout is considered adequate to cover all costs associated with the disconnection.

It is beyond the scope of this report to thoroughly address the legal implications of downspout disconnections. It is suffice to say that other municipalities (Township of Sandwich West 1962) have passed legislation forcing homeowners to disconnect downspouts from draining directly into storm sewers. The Stratford/Avon River report (Reference 3-1) stated that a voluntary program, based on public education and technical assistance, would likely be the most cost-effective approach to implementation of such

TABLE 3-6

CITY OF YORK

DOWNSPOUT SURVEY

Street	Total No. Houses	Total No. Downspouts	Downspouts Discharging To Sewer	Downspouts Discharging To Surface	Downspouts Discharging To Sewer At Front
Chiltern Hill Road	55	157	140	17	79
Ridelle Avenue	51	114	98	16	72
Kirknewton Avenue	99	212	193	19	101
Glenholme Avenue	110	251	209	42	125
Montcalm Avenue	59	130	109	21	65
Juliet Crescent	70	154	138	16	115
Silverthorn Avenue	45	101	86	15	51
McRoberts Avenue	130	301	247	54	158
Totals	619	1,420	1,220	200	766
Average Number of Downspouts Per Dwelling		2.3	86%	14%	63%

TABLE 3-7

CITY OF YORK - DOWNSPOUT DISCONNECTIONS

COST SUMMARY

Area	(1) No.of Dwelling Units	(2) Total No. Downspouts @ 2.3/unit	(3) No. of Downspouts Discharging To Sewer 86% of (2)	(4) No. of Front Downspouts 63% of (3)	Disconnect. Costs @ $100 per Downspout
Western	6,910	15,893	13,668	8,610	$ 861,000
Central	9,280	21,344	18,356	11,564	$1,156,400
Eastern	3,630	8,349	7,180	4,523	$ 452,300
Totals	19,820	45,586	39,204	24,697	$2,469,700
				SAY	$2,500,000

measures. In the case of the City of York, a method consisting of a disconnection by-law in conjunction with a public awareness program, where all work is carried out by the City, could be the most logical means of achieving City-wide disconnection.

3.4 Source Controls for Pitched Roofs

As an alternative to downspout disconnection, source controls (flow regulators) may be used to control runoff from pitched roofs. One device that is currently available is a specially designed orifice plate that fits into the downspout opening in the eavestrough. The orifice allows a fixed steady flow to be conveyed to the combined sewer. When the capacity of the orifice is exceeded, the roof runoff will overflow the eavestrough and be conveyed overland to the nearest outlet.

The regulator currently available restricts the flows from the roofs to that of a one year storm event. Our analysis indicates that this device would have little effect in reducing combined sewer overflows for the less intense rainfall events, and limited impact on reducing basement flooding for those areas where detention is being considered. However, as stated in Task 2 (see page 2-23), there are specific locations throughout the City of York where such a device could achieve the desired results as an alternative to downspout disconnection.

TABLE 3-8

COMBINED SEWER CATCHMENT DATA

Catchment	Catchment Area (ha)	Percent Impervious	Roof Area (ha)	Number of Homes To Surface
Strathearn Rd	114.2	19	9.1	2030
Arlington Ave	80.6	19	6.4	1600
Roselawn Ave	78.7	19	6.3	1300
Rogers Rd	220.0	19	17.6	3910
Hyde Ave	127.1	19	10.2	1920
Rotherham Ave	62.2	19	5.0	1190
Weston Rd	147.7	59	36.6	1680
Cripps Ave	30.6	19	2.4	530
Eileen Ave	191.9	33	26.6	4700
Summit Ave	46.0	19	3.7	960

Note: Catchment areas are shown on Plate 3-3 and Plate 3-4

4 - COMBINED SEWER CATCHMENT DATA

The following catchment data was obtained for each combined sewer catchment:

(i) Catchment area

(ii) Percent impervious of the catchment

(iii) Number and total area of roofs directly connected to the combined sewer system.

The catchment boundaries (Plates 3-3 and 3-4) were taken from the Gore and Storrie report. The areas were adjusted to include sewer separation which has taken place up until 1983. The Gartner Lee report (Reference 3-2) was used to estimate the percent impervious of each catchment. Each land use, as given in the Gartner Lee report, was used in determining the the percent impervious.

The number of roof downspouts connected to the combined sewer system has previously been discussed in Section 3.3. In total 619 homes and 1440 downspouts were included in the survey. Table 3-6 summarizes the field information for the roof downspout survey.

For each catchment area a sample area was selected. The total number of houses (roofs) within the sample area was then determined. The total number of homes in the catchment area was then established, using the ratio of residential area within the catchment to residential area of the sample area.

The area of roofs was estimated by selecting sample areas and estimating the percent roof area divided by the total impervious area. This value, on average, was found to be equal to 0.42. Table 3-8 summarizes the catchment area data.

5 - CONCLUSIONS

1. Approximately $17,250,000 is required to complete the remaining trunk storm sewers in the City of York's sewer separation scheme. The completion of the local storm sewers require funding in the amount of $67,450,000, for a grand total of $84,700,000. An additional $1,800,000 will be required to provide remedial works in the Westmount -Pinewood Area.

2. The costs of providing individual storm water connections to serve each lot are a prohibitive $127,000,000. Only the front downspouts would be connected for this sum.

3. The costs to disconnect all front downspouts in the City and discharge roof runoff to the surface will cost approximately $2,500,000.

REFERENCES

1. "Stratford/Avon River Environmental Management Report - Municipal Experience In Inflow Control Through Removal Of Household Roof Leaders" Prepared by T. Crozier and M. Niedbala, January 1984.

2. Gartner Lee Associates Limited "TAWMS Task 3 - Storm Sanitary and Combined Sewer Mapping and Data Enumeration", A draft report to the Ministry of the Environment, July 1983

APPENDIX 3-1

WESTERN AREA - COST BREAKDOWN

A. Trunk Sewers

W1	York	$ 136,300
W2	Rockcliffe, Lambton, Guestville, Dennis	1,154,000
W3	Outlook	166,400
W4	Avon North	187,200
W5	Avon South	178,200
W6	Cayuga	217,800
W7	Ellins, Foxwell	374,800
W8	Rockcliffe, Castleton, Henrietta, St. Clair, Jane, Corbett	3,060,000
W9	Willard	103,200
W10	Windermere, Beresford	181,800
Sub-Total:		$5,759,700
Scheduled for 1985 Construction:		
W11	Scarlett Road outlet	1,600,000
Total, Trunk Sewers		$7,359,700

B. Local Sewers @ $605.00 per metre

To Trunk W1	York Avenue and related drainage area	1,950 m	$1,179,750
W2	Rockcliffe North and related drainage area	2,560 m	1,548,800
W3	Outlook and related drainage area	1,860 m	1,125,300
W4	Avon North and related drainage area	820 m	496,100
W5	Avon South and related drainage area	1,430 m	865,150
W6	Cayuga and related drainage area	550 m	332,750
W7 & 11	Ellins, Foxwell and related drainage area	2,010 m	1,216,050
W8	Rockcliffe, south and related drainage area	4,110 m	2,486,550
W9	Willard and related drainage area	3,140 m	1,899,700
W10	Windermere, Beresford and related drainage area	2,320 m	1,403,600
W12	Existing Buttonwood outlet and " " "	2,300 m	1,391,500
W13	Existing industry outlet and " " "	1,580 m	955,900
W14	Existing Weston Road outlet and " " "	850 m	514,250
W15	Existing Symes outlet and related drainage area	430 m	260,150
W16	Existing Haney outlet	380 m	229,900
W17	Existing Bernice	170 m	102,850
Sub-Total:			$16,008,300
Local Sewers, Scheduled for 1985 construction, Lambton Avenue and Brownville Avenue from Weston Road to 150 m north of Weston Road			92,000
Total, Local Sewers:			$16,100,300

APPENDIX 3-2

CENTRAL AREA - COST BREAKDOWN

A. Trunk Sewers

Street	From	To	Cost	
Leg C1.				
Keele	Irving	Beechborough	191,700	
Beechborough	Keele	Bertram	117,200	
Beechborough	Bertram	Glenhaven	275,800	
Beechborough	Glenhaven	W. of Woodborough	314,300	
Castlefield	W. of Woodborough	Kincort	108,900	
Castlefield	Kincort	Carnarvon	73,800	
Castlefield	Carnarvon	C.N.R.	139,800	
Castlefield	C.N.R.	Caledonia	180,500	
Castlefield	Caledonia	Ronald	187,300	
Total, Leg C1			$1,589,300	$1,590,000
Leg C2.				
Eglinton	Trethewey	Keele	$ 72,900	
Keele	Eglinton	York	180,000	
York	Keele	Glenhaven	405,500	
Shopping Plaza	Glenhaven	Carnarvon	131,800	
C.N.R.	Carnarvon	Croham	121,300	
Bowie	Croham	Caledonia	103,500	
Total, Leg C2			$1,015,000	$1,015,000
Leg C3.				
Dufferin	Rogers	Holmesdale	$1,040,000	
Dufferin	Holmesdale	Preston	286,500	
Dufferin	Preston	Easement	62,300	
Total, Leg C3			$1,388,800	$1,390,000
		Sub-Total:		3,995,000

APPENDIX 3-2

CENTRAL AREA - COST BREAKDOWN

A. Trunk Sewers - cont...

Street	From	To	Cost	
Leg C4.				
Ypres	Keele	Kane	$ 192,000	
Kane	Ypres	Eglinton)		
		)	195,000	
Eglinton	Kane	Easterly)		
Total, Leg C4			$ 387,000	$ 387,000

Leg C4 is scheduled for 1985 construction. The cost for this leg of storm trunk sewer installation has been taken from the City of York Department Rankiong of 1985 Capital Projects. Leg C4 includes rank items 8 and 9.

Total, Trunk Sewers		$4,382,000

B. Local Sewers @ $605 per metre

To Trunk C1 Yorktown outlet	1,310 m	$ 792,550	
C2 Eglinton outlet	3,200 m	1,936,000	
C3 Dufferin	11,460 m	6,933,300	
C4 Rotherham outlet	1,310 m	792,550	
C5 Existing Hyde outlet	30,720 m	18,585,600	
Sub-Total:		$29,040,000	$29,040,000

Local Sewers, Scheduled for 1985 construction:

Hopewell-Lockley to Times	$ 110,000	
Roselawn-Miranda to Dufferin	234,000	
Castlefield-Miranda to Dufferin	245,000	
Ridelle-Dufferin to Lockley	92,000	
Ewart-Keele to Scott	69,000	
Trowell-Keele to Harleton	260,000	
Donald-Keele to Haverson	300,000	
Sub-Total:	$1,310,000	$1,310,000

Total, Local Sewers		$30,350,000

APPENDIX 3-3

EASTERN AREA - COST BREAKDOWN

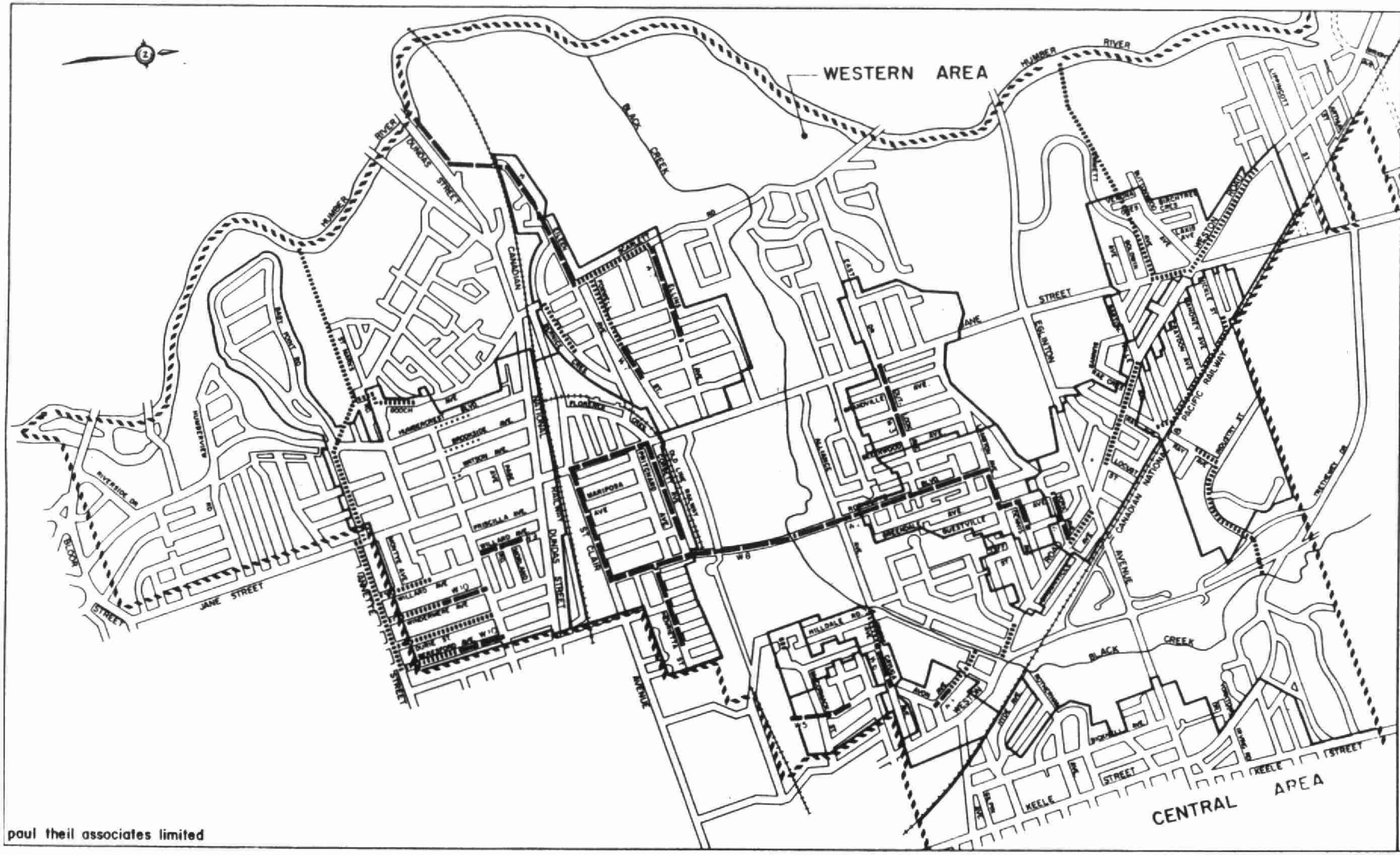
A. Trunk Sewers

Street	From	To	Cost
Easement	Ex. Trunk	Strathearn	$ 178,200
Strathearn	Easement	Markdale	277,900
Markdale	Strathearn	Glencedar	314,300
Glencedar	Markdalen	Dewbourne	835,000
Dewbourne	Glencedar	Camberwell	180,300
Camberwell	Dewbourne	Easement	144,300
Easement	Camberwell	Bathurst	780,000
Total, Trunk Sewers:			$2,710,000

B. Cost Estimate, Local Sewers

Local Sewers @ $605 per metre

To Trunk 1 Prop. trunk, Strathearn system	8,200 m	$4,961,000
2 Existing Valewood system	1,600 m	968,000
3 Existing Durham system	5,900 m	3,569,500
Sub-Total:		$9,498,500
Local Sewers, Scheduled for 1985 construction:		
Vaughan-Oakwood to Glenora		$46,000
Total, Local Sewers:		$9,544,500



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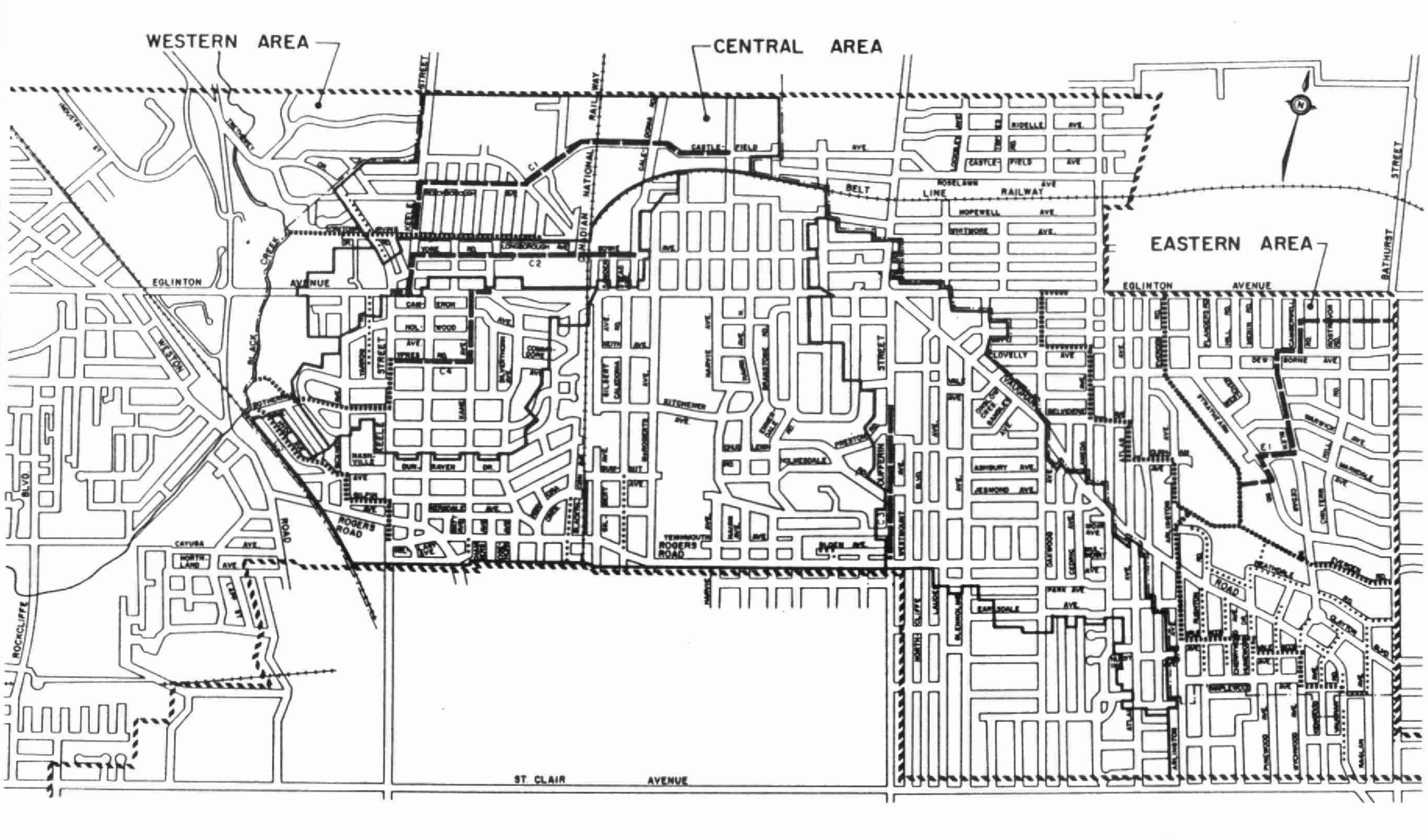
Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

EXISTING AND PROPOSED TRUNK STORM SEWERS Western Area

legend



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Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

EXISTING AND PROPOSED TRUNK STORM SEWERS

Central And Eastern Areas

scale 0 100 300 500 700 metres

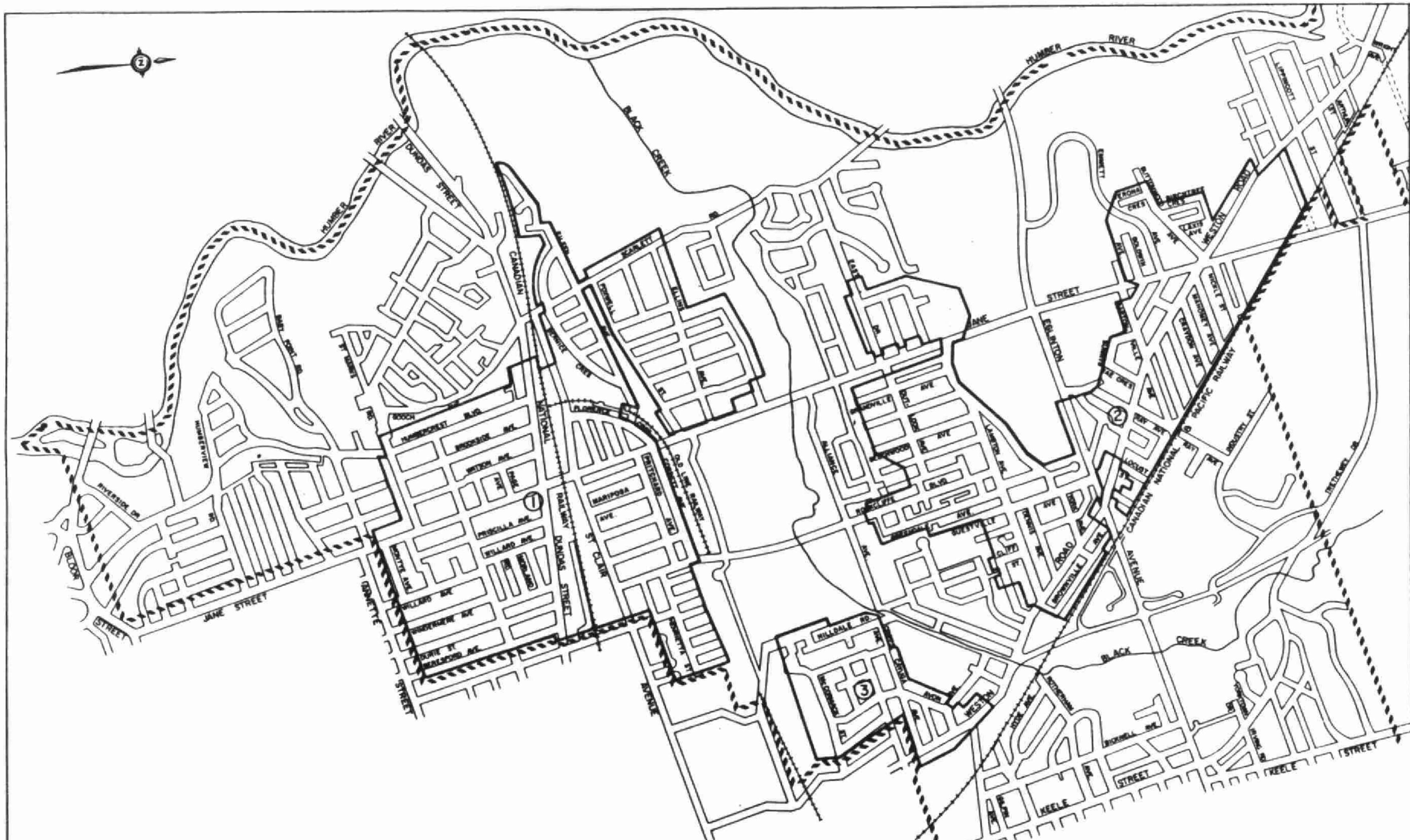
legend

- city limits
- area boundaries
- existing trunk storm sewer
- existing storm sewer (as per 1967 Gore & Storrie report)
- proposed trunk storm sewer

NOTE: Sewer system at end of 1984

- district boundary

plate 3.2



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Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

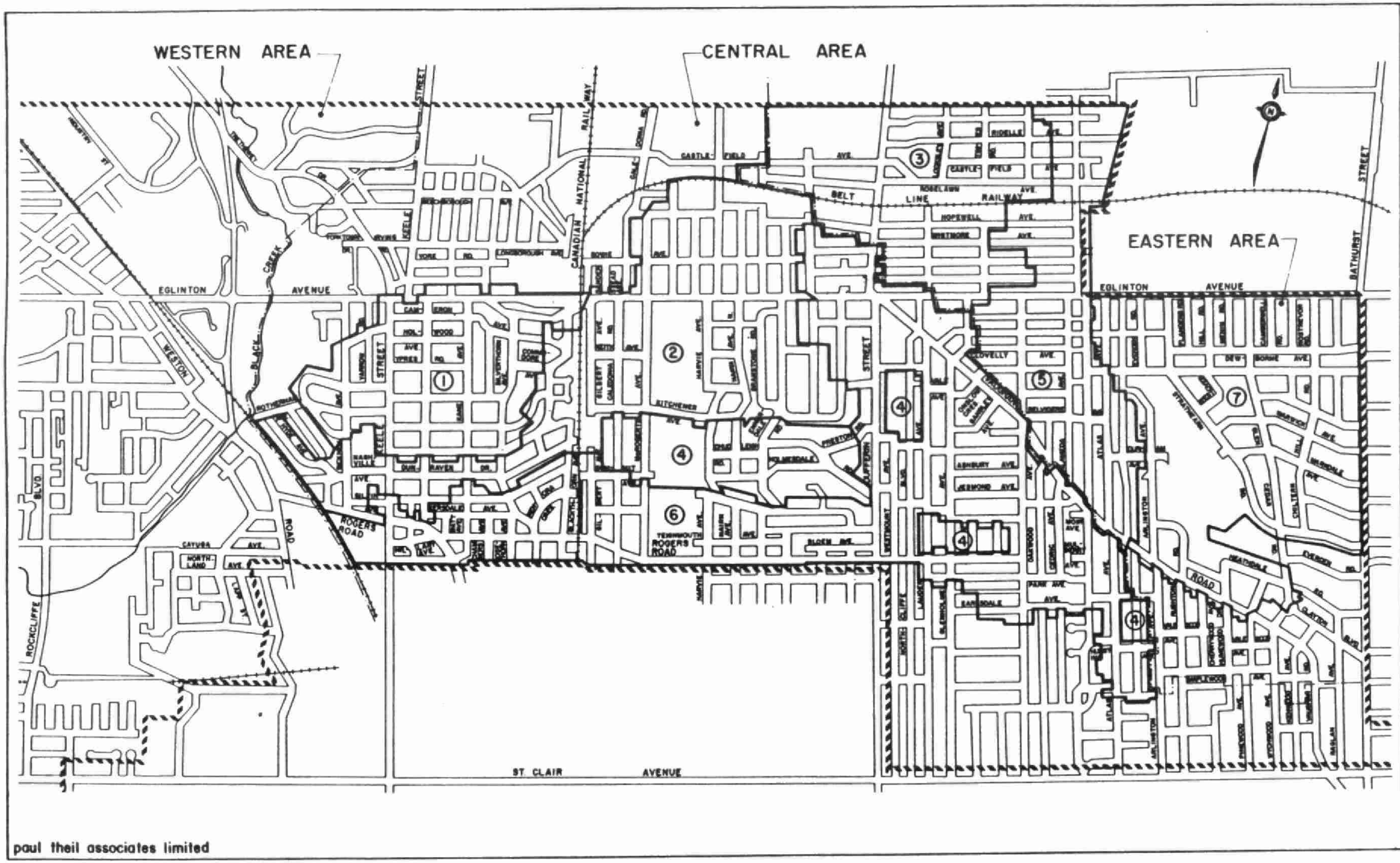
COMBINED SEWER CATCHMENTS Western Area

0 100 300 500 700

scale metres

legend

NO	CATCHMENT
1	Eileen
2	Weston
3	Cripps



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

COMBINED SEWER CATCHMENT
Central And Eastern Areas

legend

NO	CATCHMENT
1	Rotherham
2	Hyde
3	Roselawn
4	Summit
5	Arlington
6	Rogers
7	Strathearn

— — — District Boundary

plate 3-4

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SECTION 4

TASK 4

Stormwater Detention Ponds

TASK 4

STORM WATER RETENTION PONDS

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FEASIBILITY STUDY AND COSTING OF
PROPOSED POLLUTION CONTROL MEASURES
IN THE HUMBER WATERSHED

TASK 4

STORM WATER RETENTION PONDS

1 - INTRODUCTION

A separate TAWMS study has been undertaken to determine the effective use of retention ponds to control the pollution of stormwater runoff to the Humber River and its tributaries. From that study, major catchment areas containing extensive industrial and commercial development were identified.

Task 4 consists of an engineering feasibility study and cost estimates of constructing retention ponds at or near the point of storm sewer outlets to the Humber or its tributaries from four such catchment areas.

2 - STUDY METHODOLOGY

The initial approach to this task consisted of determining the catchment areas for several industrial areas as identified by the Ministry of the Environment. A total of eight sites, five in North York and three in Etobicoke, were reviewed to establish the potential of providing sufficient retention volumes. As an objective, a criteria of 20 mm average runoff over the catchment area was specified by the Ministry as the desirable volume of the retention ponds.

A schedule of volumes for each site and relationship to the design criteria was prepared and submitted to the M.O.E. Based on that, we were advised that detailed study and costing for four sites were to be undertaken, as initially envisaged. Three of these were in Etobicoke and one in North York, at the locations shown on Plate 4-1. The catchment areas include some residential areas, which could not practically be separated from the storm sewer system.

In addition to the objectives stated in the terms of reference, M.O.E requested by letter of April 3, 1985 that the following be accounted for in the study of the storm water retention ponds.

1. To ascertain and demarcate the pond's full catchment.

2. To ascertain the feasibility of diverting, wholly or partly, the influent to by-pass the pond under intense storm conditions to avoid washout of pond sediments.

3. To ascertain the conditions under which a pond, if it is instream, may be topped over by stream flood water.

4. To see to the need for vehicular access to a pond for maintenance and de-sludging.

5. To have consultation with official agencies concerned respecting the prospective use of the land for a storm water pollution control pond.

These matters have been addressed in this study.

3 - GENERAL DESIGN CRITERIA

In completing the design of the proposed facilities, many factors were taken into account, the more significant being listed below with comments.

1. Excavation requirements and the ease of carrying out the excavation works.

2. Slope stability assessment. Detailed analysis will need to be undertaken where ponds are adjacent to residential areas. Erosion is not anticipated to be a problem.

3. The location and depth of existing storm and sanitary sewers. In some cases, existing manhole tops will have to be raised. The works will in some cases encroach upon some existing easements. Approval from the respective authorities will be required .

4. The continuation of use for existing park facilities under dry weather conditions.

5. The inverts of upstream sewers. The impact of the proposed facilities on upstream sewers was considered. A detailed analysis may be required for some of these at the detail design stage.

6 The impact of the potential for increased water levels due to the construction of the proposed facilities. Discussions were held with M.T.R.C.A. staff. It was agreed that at the detail design stage, a computer analysis may be required to confirm the impact of the proposed facility on the Regional Flood level.

7. The feasibility of diverting the inflow around a storage facility during the maintenance period. It was determined that, due to lack of available land, diversion of the inflow during times when sediments are being removed would not be feasible. Provided ponds are cleaned during reasonably dry periods, this should not represent any problems.

8. The selection of the normal water level and the relationship between the normal water level, maximum water level, pond outlet size and spillway size. The normal water level and outlet pipe size were selected on a preliminary basis to provide sufficient volume to allow settlement of particles greater than 100 microns in diameter (Reference 1) for both the more frequent storms and the maximum design storm.

9. Access to trash racks for maintenance.

10. All spillways were designed to pass the 100 year storm flows.

11. Cut-off walls at outlet pipes to prevent outflow through pipe bedding and trenches.

12. Maintenance of ponds including capability of access ramps to support maintenance equipment.

At the detail design stage several factors, which will likely influence the design of the outlet works and the selection of the normal and maximum water levels should be considered further. These include:

-The type and size of sediment to be settled in the proposed facility.

-The selection of design storms and the routing of the runoff flows through the proposed facilities. This will likely require a detailed hydraulic analysis and will influence the final size of the facilities.

-The potential of an increase in the Regional flood levels due to the construction of a proposed facility.

-The provision of individual cells or short circuit devices.

The above factors were not considered during this study, as sufficient data was not available.

4 - SUMMARY OF FINDINGS

The four locations at which the feasibility of constructing retention ponds has been studied are shown on the location plan (Plate 4-1) and described as follows:

- North York: St. Lucie Drive and Storer Drive
- Etobicoke: Martingrove Road and West Humber Boulevard
- Etobicoke: Albion Road and Norfield Crescent
- Etobicoke: Islington Avenue and Saskatoon Drive

The catchment areas and corresponding retention volumes, based on 20, 15 and 10 mm runoff depths per ha, are listed in Table 4-1. Summary descriptions of the study findings for each of these sites follow.

4.1 North York: St. Lucie Drive and Storer Drive (Plates 4-2 and 4-3)

The pond is proposed west of Weston Road and south of Finch Avenue, in the floodplains for Emery Creek on a tributary to the Humber River. The pond is divided into two cells to avoid conflict with an existing gas pipe line and H.E.P.C. towers.

The pond, as shown on Plate 4-2, has a capacity of 154,000 m^3, equivalent to a depth of runoff of 20 mm over the 770 ha drainage area. To avoid flooding of the lands over the pipe line and avoid the towers, the normal water level for retention should not exceed elevation 126.0.

During times when the full 20 mm runoff depth occurs, the water will rise to elevation 127.8, which is the spillway level as shown on section B-B, Plate 4-3. In actual fact this level is on the conservative side, since we have not allowed for the discharge that will occur through the downpipe controlling the normal water level. To provide maximum retention time, this pipe should be relatively small. As part of detailed design, a number of storms should be modelled for routing through the pond and the spillway elevation lowered accordingly. All storm sewer outlets contributing to this pond are above the high water level.

TABLE 4-1

CATCHMENT AREAS AND RETENTION VOLUMES

			Runoff Depth per ha (mm)		
			20	15	10
Pond Location	Catchment Area (ha)	Indus. Area (ha)	Volume (m^3)	Volume (m^3)	Volume (m^3)
North York -St. Lucie & Storer	770	650	154,000	115,500	77,000
Etobicoke -Martingrove & West Humber	264	168	52,800	39,600	26,400
-Albion & Norfield	480	160	96,000	72,000	48,000
-Islington & Saskatoon	650	330	130,000	97,500	65,000
TOTAL	2,164	1,308	432,800	324,600	216,400

The two cells will be interconnected with a pipe across the space for the pipe line, and discharge for flows exceeding the normal water level will be provided through a downpipe as shown schematically on section B-B,Plate 4-3.

Access to the site is available from Lanyard Avenue at Weston Road.

4.2 Etobicoke: Martingrove Road and Westhumber Boulevard (Plates 4-4 and 4-5)

To accommodate the 20 mm depth of runoff over the 264 ha catchment area, discharging to the west branch of Humber River at Martingrove Road, a total pond volume of 52,800 m^3 is required. This can only be achieved by creating two ponds, one either side of Martingrove Road, as shown on Plates 4-4 and 4-5.

To minimize the amount of earthwork, the high water level for the westerly pond should be set at elevation 149.0, and at elevation 148.1 for the easterly pond. The normal water level has been selected to be below the sanitary manholes to allow access to the sewer during dry weather periods.

4.3 Etobicoke: Albion Road and Norfield Crescent (Plates 4-6 and 4-7)

The storm sewers discharging to the small tributary to Humber River west of Albion Road serve a drainage area of about 480 ha. For a runoff depth of 20 mm, the required retention volume is 96,000 m^3. Plate 4-6 shows a configuration for a pond with this volume. It is created by berming across the small tributary and by excavating within the floodplain upstream of the berm. Access can be obtained from Albion Road.

In order to provide access to manholes on the two sanitary sewers at the west bank of the tributary, the normal water level should not exceed elevation 124.0. This level can be maintained with a downpipe arrangement as shown on Section B-B, Plate 4-7. The high water level corresponding to 20 mm runoff depth will be 126.0, being the level of the spillway.

4.4 Etobicoke: Islington Avenue and Saskatoon Drive (Plates 4-8 and 4-9)

The catchment area discharging to the small tributary to Humber River north of Saskatoon Drive and east of Islington Avenue covers approximately 650 ha. The required retention volume to accommodate the 20 mm depth of runoff is 130,000 m^3. This volume can be obtained by building a berm across the tributary approximately 350 metres east of Islington Avenue.

In order to maintain the use of the tennis courts at the southwest corner of Dixon Road and Islington Avenue and provide access to all sanitary manholes, the normal water level should be kept at or below elevation 141.0. To accommodate the total runoff depth of 20 mm, the maximum water level will rise to elevation 143.0. Higher runoff volumes will result in overflow over the spillway directly to the existing open watercourse.

The general criteria and constraints which influence the design of each pond have been summarized in Table 4-2.

TABLE 4-2

POND SELECTION FACTORS AND CRITERIA

Pond Location	Pond Location Limits	Pond N.W.L.	Pond H.W.L.	Pond Invert	Spillway Sizing	Access	Misc.
North York -St. Lucie & Storer	-Ex. embankments -Ex. tree stands -Humber River -Hydro towers -Pipeline	-Hydro towers & pipeline kept above NWL for maintenance -Access requirements -Humber River water elevations -Ex. storm outlet	-Set by capacity requirement 20mm x drainage area -100 year flow in Humber River	-Ex. river elevation for draining of pond -Ex. sewers -Economics	-100 year storm	-From Lanyard Road through ex. valley	-Twin Ponds are balanced by control piping
Etobicoke -Martingrove & West Humber	-Ex.valley shape -Trees -Humber River -Valley access	-Humber River water elevations -Access requirements	-Set by capacity requirement 20mm x drainage area -100 year flow in Humber River	-Ex. river elevation for draining of pond -Economics	-100 year storm	-From Martingrove Road	-Water levels are controlled by piping under Martingrove -Existing pedestrian access relocated
-Albion & Norfield	-Ex.embankment -Tree stand -Albion Road -Access	-Access to existing manholes -Humber River water elevations	-Set by capacity requirement 20mm x drainage area -100 year flow in Humber River	-Ex. river elevation for draining of pond -Ex. sewers -Economics	-100 year storm	-From Albion Road	
-Islington & Saskatoon	-Ex.valley shape -Ex.stm.outlets	-Ex.stm.outlet -Ex. manholes -Tennis courts -Valley use	-Ex. properties -Islington Ave.	-Ex. downstream elev. for draining of pond -Ex.sewers -Economics	-100 year storm	-From Saskatoon Drive	

5 - ESTIMATE OF COSTS

5.1 Capital Costs

Cost estimates for the storm water retention ponds are presented below. The estimates were based on 1984 construction costs and include a 20 percent engineering and contingency cost.

Earth moving was calculated on the basis of the most recent topographic maps and the proposed dimensions shown on the drawings herein. Surplus excavation has been assumed to be removed by trucking. Major earthworks are assumed to be carried out between June and September. The ponds are assumed to be unlined. Land costs were not included as all subject lands are owned by various government agencies.

In addition, allowances have been made for:

- restoration of areas affected by construction
- chain link fencing
- outlet piping and valves
- raising existing manholes tops and changing to watertight covers
- spillways
- access roads

Cost estimates have been prepared for ponds sized to accommodate either 10 mm, 15 mm or 20 mm of runoff depth per ha. A breakdown of the major cost items are presented in the appendix to this section.

Various parameters influence the cost, such as:

- retention capabilities of the natural valley shape which in turn determines the amount of excavation and earth to be moved.
- The extent of utility relocation and new piping.
- The length of access road required to the ponds.
- The extent of restoration required.

A summary of the cost estimates is presented in Table 4-4. The least costly alternative (based on cost per hectare of catchment area) for each location has been underlined.

A maintenance cost estimate has been detailed in Table 4-3 and included in the cost summary, Table 4-4.

5.2 Maintenance Cost

An estimate of the following main items of maintenance has been prepared:

1. Maintenance of grassed embankments

   Due to the fact that the cost of cutting grass of the pond embankments is offset by the present grassed area that will be covered by water, no allowance has been made for additional cost.

2. Weed and algae control.

   To reduce weed and algae growth a minimum pond depth of 1.5 metres is suggested. We consider this cost to be minor, but have allowed and annual cost of $100 per 10,000 m^3 of pond volume.

3. Monitoring

   Periodic monitoring of water quality should be carried out. An allowance of $40 per 10,000 m^3 pond volume has been made.

4. Inspection and litter removal

   This will be a part of normal park maintenance and should not be much different to current costs. An annual allowance of $100 per 10,000 m^3 is suggested.

TABLE 4-3

ESTIMATE OF MAINTENANCE COSTS

	North York	Etobicoke			
	St. Lucie & Storer	Martingrove & West Humber	Albion & Norfield	Islington & Saskatoon	TOTAL
Solids per km of curb per annum: m^3	31.84	31.84	31.84	31.84	
Length of curb in drainage area: km	84.00	34.10	66.00	75.50	
Solids to pond (30% of total): m^3	802	326	630	721	
0.5 m average sediment depth: m^3 *	18,000	11,000	8,000	5,000	
Years between cleaning of 0.5 m aver. depths *	22.4	33.7	12.7	6.9	
Annual cost of cleaning based on $12 per m^3	$9,643	$3,917	$7,559	$8,696	$29,815
Weed and Algae Control *	1,540	528	960	1,300	4,328
Water quality monitoring *	620	210	380	520	1,730
Inspection and litter removal *	1,540	528	960	1,300	4,328
	13,343	5,183	9,850	11,816	40,201
Present Value Cost for Maintenance 20 mm storage					562,814
15 mm storage					526,462
10 mm storage					490,034

* Based on 20 mm runoff per ha.

TABLE 4-4

COST SUMMARY

	Runoff per ha (mm)					
	20 mm	20 mm	15 mm	15 mm	10 mm	10 mm
Location	Estimated Cost	Cost per ha of C'ment Area	Estimated Cost	Cost per ha of C'ment Area	Estimated Cost	Cost per ha of C'ment Area
North York -St. Lucie & Storer	$1,184,000	$ 1,538	$ 870,000	$ 1,130	$ 440,000	$ 571
Etobicoke -Martingrove & West Humber	$ 841,000	$ 3,186	740,000	2803	473,000	1,792
-Albion & Norfield	$ 795,000	$ 1,590	553,000	1,110	382,000	796
-Islington & Saskatoon	$ 425,000	$ 654	312,000	480	301,000	463
TOTAL including 20% Eng. & cont.	$3,245,000		$2,475,000		$1,596,000	
Average Cost per Catchment ha		$ 1,500		$ 1,144		$ 738
Present Value Cost for Operation and Maintenance	562,814		526,462		490,034	
TOTAL COST	3,777,814		3,001,462		2,086,034	

Most cost effective alternative on a unit basis for each facility is underlined

5. Removal of sediments

An estimate of the volume of solids that can be expected to be washed off the paved streets within the watershed of each of the ponds is presented in Table 4-3. These volumes are based on data from E.P.A. publication R2-72-081 "Water Pollution Aspects of Street Surface Contaminants". We have assumed normal street sweepng will be carried out, which according to the E.P.A. study will remove about 50 percent of the solids. The study also states that a further 20 percent of the solids is collected in catchbasin sumps, sewers and manholes, or remains on the streets, leaving 30 percent of the total solids to end up in the detention ponds.

REFERENCES

1. "Practices in Detention of Urban Storm Water Runoff", American Public Works Association, Special Report No. 43, June 1974

COST BREAKDOWN

STORM RETENTION POND - 20 mm RUNOFF DEPTH

NORTH YORK - ST. LUCIE DRIVE AND STORER DRIVE (PLATES 4-2 AND 4-3)

Excavation - West Pond	72,000 m^3
- East Pond	32,000 m^3
Total:	104,000 m^3

Excavate and use on site	28,000	m^3	@	$ 2.25	$ 63,000.00
Excavate and dispose	76,000	m^3	@	5.00	380,000.00
Piping			LUMP SUM		95,000.00
Spillways	4,300	m^2	@	35.00	150,500.00
Access ramps	161	m^2	@	21.00	3,381.00
Construct earthen dam	28,000	m^3	@	2.10	58,800.00
Access road	4,000	m^2	@	5.00	20,000.00
Fencing	1,500	m	@	30.00	45,000.00
Restoration	57,200	m^2	@	3.00	171,000.00
Sub-Total:					$ 986,681.00
Engineering & Contingencies 20%					197,319.00
Total:					$1,184,000.00

Cost per hectare of Catchment Area	=	$ 1,538.00
Cost per hectare of Industrial Catchment Area	=	$ 1,821.00

COST BREAKDOWN

STORM RETENTION POND - 20 mm RUNOFF DEPTH

ETOBICOKE - MARTINGROVE ROAD AND WEST HUMBER BOULEVARD (PLATES 4-4 AND 4-5)

Excavation - West Pond 40,700 m^3
- East Pond 26,100 m^3

Total: 66,800 m^3

Excavate and use on site	43,700	m^3	@	$ 2.25	$ 98,325.00
Excavate and dispose	23,100	m^3	@	5.00	115,500.00
Piping			LUMP SUM		65,000.00
Sanitary sewer	95	m	@	333.00	31,635.00
Spillways	3,300	m^2	@	35.00	115,500.00
Clean out ramps	724	m^2	@	21.00	15,204.00
Access roads	3,000	m^2	@	$18.00	54,000.00
Reconstruct asphalt walkway	1,220	m^2	@	18.00	21,960.00
Fencing	1,200	m	@	30.00	36,000.00
Construct earthen dam	43,700	m^3	@	2.10	91,770.00
Restoration	18,500	m^2	@	3.00	55,500.00
Sub-Total:					$700,394.00
Engineering & Contingencies 20%					140,606.00
Total:					$841,000.00

Cost per hectare of Catchment Area = $ 3,186.00

Cost per hectare of Industrial Catchment Area = $ 5,006.00

COST BREAKDOWN

STORM RETENTION POND - 20 mm RUNOFF DEPTH

ETOBICOKE - ALBION ROAD AND NORFIELD CRESCENT (PLATES 4-6 AND 4-7)

PLAN #E-2

Total Excavation - 55,000 m^3

Excavate and use on site	14,000	m^3	@	$ 2.25	$ 31,500.00
Excavate and dispose	41,100	m^3	@	5.00	205,500.00
Piping			LUMP SUM		20,000.00
Spillway	4,000	m^2	@	35.00	140,000.00
Access ramp	190	m^2	@	21.00	3,990.00
Access road	1,700	m^2	@	$18.00	30,600.00
Fencing	700	m	@	30.00	21,000.00
Construct earthen dam	14,000	m^3	@	2.10	29,400.00
Restoration	59,000	m^2	@	3.00	177,000.00
Raise existing manholes	4	ea.	@	800.00	3,200.00
Sub-Total:					$662,190.00
Engineering & Contingencies 20%					132,810.00
Total:					$795,000.00

Cost per hectare of Catchment Area	=	$ 1,590.00
Cost per hectare of Industrial Catchment Area	=	$ 4,769.00

COST BREAKDOWN

STORM RETENTION POND - 20 mm RUNOFF DEPTH

ETOBICOKE - ISLINGTON AVENUE AND SASKATOON DRIVE (PLATES 4-8 AND 4-9)

Total Excavation - 28,400 m^3

Excavation	15,000	m^3	@	$ 2.25	$ 33,750.00
	13,400	m^3	@	5.00	67,000.00
Piping				LUMP SUM	20,000.00
Spillway	4,000	m^2	@	35.00	140,000.00
Access ramp	200	m^2	@	21.00	4,200.00
Access road	600	m^2	@	$18.00	10,800.00
Fencing	900	m	@	30.00	27,000.00
Construct earthen dam	15,000	m^3	@	2.10	31,500.00
Construct proposed headwall	1	ea.	@	8,125.00	8,125.00
Remove existing storm pipe	30	m	@	100.00	3,000.00
Adjust 2 sanitary manholes					2,000.00
Restoration	2,200	m^2	@	3.00	6,600.00
Sub-Total:					$353,975.00
Engineering & Contingencies					71,025.00
Total:					$425,000.00

Cost per hectare of Catchment Area = $ 654.00

Cost per hectare of Industrial Catchment Area = $ 1,288.00

<u>COST BREAKDOWN</u>

<u>STORM RETENTION POND - 15 mm RUNOFF DEPTH</u>

<u>NORTH YORK - ST. LUCIE DRIVE AND STORER DRIVE (PLATES 4-2 AND 4-3)</u>

Excavation - West Pond	33,500 m^3
- East Pond	32,000 m^3
Total:	65,500 m^3

Excavate and use on site	28,000	m^3	@	\$ 2.25	\$ 63,000.00
Excavate and dispose	37,500	m^3	@	5.00	187,500.00
Piping			LUMP SUM		95,000.00
Spillways	4,300	m^2	@	35.00	150,500.00
Access ramps	161	m^2	@	21.00	3,381.00
Construct earthen dam	28,000	m^3	@	2.10	58,800.00
Access road	4,000	m^2	@	5.00	20,000.00
Fencing	1,300	m	@	30.00	39,000.00
Restoration	36,000	m^2	@	3.00	108,000.00
Sub-Total:					\$725,181.00
Engineering & Contingencies 20%					144,819.00
Total:					\$870,000.00

Cost per hectare of Catchment Area	=	\$ 1,130.00
Cost per hectare of Industrial Catchment Area	=	\$ 1,338.00

COST BREAKDOWN

STORM RETENTION POND - 15 mm RUNOFF DEPTH

ETOBICOKE - MARTINGROVE ROAD AND WEST HUMBER BOULEVARD (PLATES 4-4 AND 4-5)

Excavation - West Pond 27,500 m^3
- East Pond 26,100 m^3

Total: 53,600 m^3

Excavate and use on site	43,700	m^3	@	$ 2.25	$ 98,325.00
Excavate and dispose	9,900	m^3	@	5.00	49,500.00
Piping			LUMP SUM		65,000.00
Sanitary sewer	95	m	@	333.00	31,635.00
Spillways	3,300	m^2	@	35.00	115,500.00
Clean out ramps	724	m^2	@	21.00	15,204.00
Access roads	3,000	m^2	@	$18.00	54,000.00
Reconstruct asphalt walkway	1,220	m^2	@	18.00	21,960.00
Fencing	1,000	m	@	30.00	30,000.00
Construct earthen dam	43,700	m^3	@	2.10	91,770.00
Restoration	14,800	m^2	@	3.00	44,400.00
Sub-Total:					$617,294.00
Engineering & Contingencies 20%					122,706.00
Total:					$740,000.00

Cost per hectare of Catchment Area = $ 2,803.00

Cost per hectare of Industrial Catchment Area = $ 4,405.00

COST BREAKDOWN

STORM RETENTION POND - 15 mm RUNOFF DEPTH

ETOBICOKE - ALBION ROAD AND NORFIELD CRESCENT (PLATES 4-6 AND 4-7)

PLAN #E-2

Total Excavation - 31,000 m^3

Excavate and use on site	14,000	m^3	@	$ 2.25	$ 31,500.00
Excavate and dispose	17,000	m^3	@	5.00	85,000.00
Piping			LUMP SUM		20,000.00
Spillway	4,000	m^2	@	35.00	140,000.00
Access ramp	190	m^2	@	21.00	3,990.00
Access road	1,700	m^2	@	$18.00	30,600.00
Fencing	600	m	@	30.00	18,000.00
Construct earthen dam	14,000	m^3	@	2.10	29,400.00
Restoration	33,000	m^2	@	3.00	99,000.00
Raise existing manholes	4	ea.	@	800.00	3,200.00
Sub-Total:					$460,690.00
Engineering & Contingencies 20%					92,310.00
Total:					$553,000.00

Cost per hectare of Catchment Area = $ 1,110.00

Cost per hectare of Industrial Catchment Area = $ 3,456.00

COST BREAKDOWN

STORM RETENTION POND - 15 mm RUNOFF DEPTH

ETOBICOKE - ISLINGTON AVENUE AND SASKATOON DRIVE (PLATES 4-8 AND 4-9)

Total Excavation - 12,000 m^3

Excavation	12,000	m^3	@	\$ 2.25	\$ 27,000.00
Piping			LUMP SUM		20,000.00
Spillway	4,000	m^2	@	35.00	140,000.00
Access ramp	200	m^2	@	21.00	4,200.00
Access road	600	m^2	@	\$18.00	10,800.00
Fencing	900	m	@	30.00	27,000.00
Construct earthen dam	12,000	m^3	@	2.10	25,200.00
Restoration	2,000	m^2	@	3.00	6,000.00
Sub-Total:					\$260,200.00
Engineering & Contingencies					51,800.00
Total:					\$312,000.00

Cost per hectare of Catchment Area = \$ 480.00

Cost per hectare of Industrial Catchment Area = \$ 945.00

COST BREAKDOWN

STORM RETENTION POND - 10 mm RUNOFF DEPTH

NORTH YORK - ST. LUCIE DRIVE AND STORER DRIVE (PLATES 4-2 AND 4-3)

Excavation - East Pond Only 27,000 m^3

Excavate and use on site	12,000	m^3	@	$ 2.25	$ 27,000.00
Excavate and dispose	15,000	m^3	@	5.00	75,000.00
Piping			LUMP SUM		31,000.00
Spillways	2,150	m^2	@	35.00	75,250.00
Access ramps	80	m^2	@	21.00	1,680.00
Construct earthen dam	12,000	m^3	@	2.10	25,200.00
Access road	4,000	m^2	@	5.00	20,000.00
Fencing	800	m	@	30.00	24,000.00
Restoration	29,000	m^2	@	3.00	87,000.00
Sub-Total:					$366,130.00
Engineering & Contingencies 20%					73,870.00
Total:					$440,000.00

Cost per hectare of Catchment Area	=	$ 571.00
Cost per hectare of Industrial Catchment Area	=	$ 677.00

COST BREAKDOWN

STORM RETENTION POND - 10 mm RUNOFF DEPTH

ETOBICOKE - MARTINGROVE ROAD AND WEST HUMBER BOULEVARD (PLATES 4-4 AND 4-5)

Excavation - West Pond Only 40,700 m^3

Excavate and use on site	22,000	m^3	@	$ 2.25	$ 49,500.00
Excavate and dispose	18,700	m^3	@	5.00	93,500.00
Piping			LUMP SUM		18,000.00
Sanitary sewer	95	m	@	333.00	31,635.00
Spillways	1,650	m^2	@	35.00	57,750.00
Clean out ramps	370	m^2	@	21.00	7,770.00
Access roads	1,200	m^2	@	$18.00	21,600.00
Reconstruct asphalt walkway	1,220	m^2	@	18.00	21,960.00
Fencing	600	m	@	30.00	18,000.00
Construct earthen dam	22,000	m^3	@	2.10	46,200.00
Restoration	9,500	m^2	@	3.00	28,500.00
Sub-Total:					$394,415.00
Engineering & Contingencies 20%					78,585.00
Total:					$473,000.00

Cost per hectare of Catchment Area = $ 1,792.00

Cost per hectare of Industrial Catchment Area = $ 2,815.00

COST BREAKDOWN

STORM RETENTION POND - 10 mm RUNOFF DEPTH

ETOBICOKE - ALBION ROAD AND NORFIELD CRESCENT (PLATES 4-6 AND 4-7)

PLAN #E-2

Total Excavation - 14,000 m^3

Excavate and use on site	14,000	m^3	@	$ 2.25	$ 31,500.00
Piping			LUMP SUM		20,000.00
Spillway	4,000	m^2	@	35.00	140,000.00
Access ramp	190	m^2	@	21.00	3,990.00
Access road	1,700	m^2	@	$18.00	30,600.00
Fencing	500	m	@	30.00	15,000.00
Construct earthen dam	14,000	m^3	@	2.10	29,400.00
Restoration	15,000	m^2	@	3.00	45,000.00
Raise existing manholes	4	ea.	@	800.00	3,200.00
Sub-Total:					318,690.00
Engineering & Contingencies 20%					63,310.00
Total:					$382,000.00

Cost per hectare of Catchment Area = $ 796.00

Cost per hectare of Industrial Catchment Area = $ 2,388.00

COST BREAKDOWN

STORM RETENTION POND - 10 mm RUNOFF DEPTH

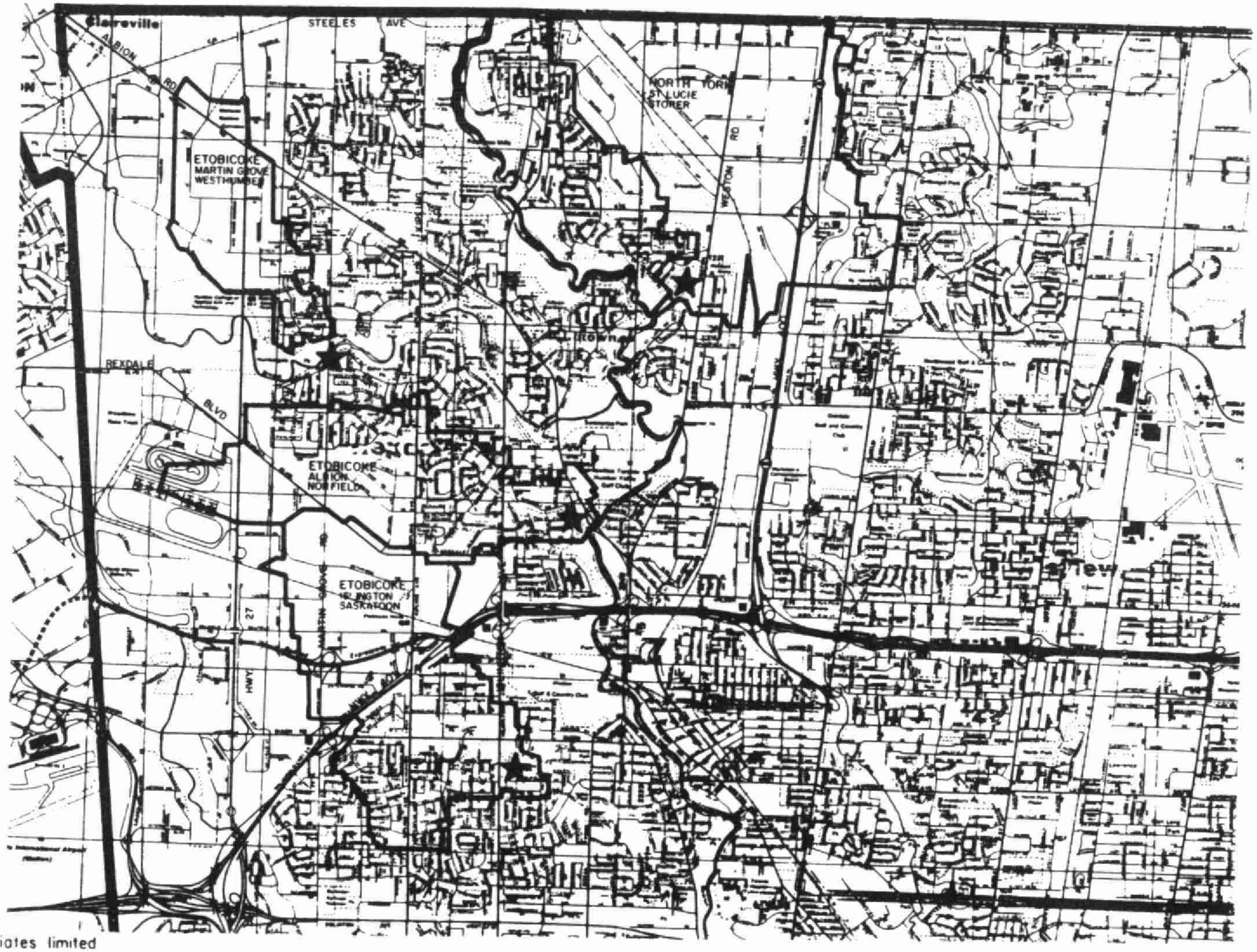
ETOBICOKE - ISLINGTON AVENUE AND SASKATOON DRIVE (PLATES 4-8 AND 4-9)

Total Excavation - 10,000 m^3

Excavation	10,000	m^3	@	$ 2.25	$ 22,500.00
Piping			LUMP SUM		20,000.00
Spillway	4,000	m^2	@	35.00	140,000.00
Access ramp	200	m^2	@	21.00	4,200.00
Access road	600	m^2	@	$18.00	10,800.00
Fencing	900	m	@	30.00	27,000.00
Construct earthen dam	10,000	m^3	@	2.10	21,000.00
Restoration	1,800	m^2	@	3.00	5,400.00
Sub-Total:					$250,900.00
Engineering & Contingencies					50,100.00
Total:					$301,000.00

Cost per hectare of Catchment Area = $ 463.00

Cost per hectare of Industrial Catchment Area = $ 912.00



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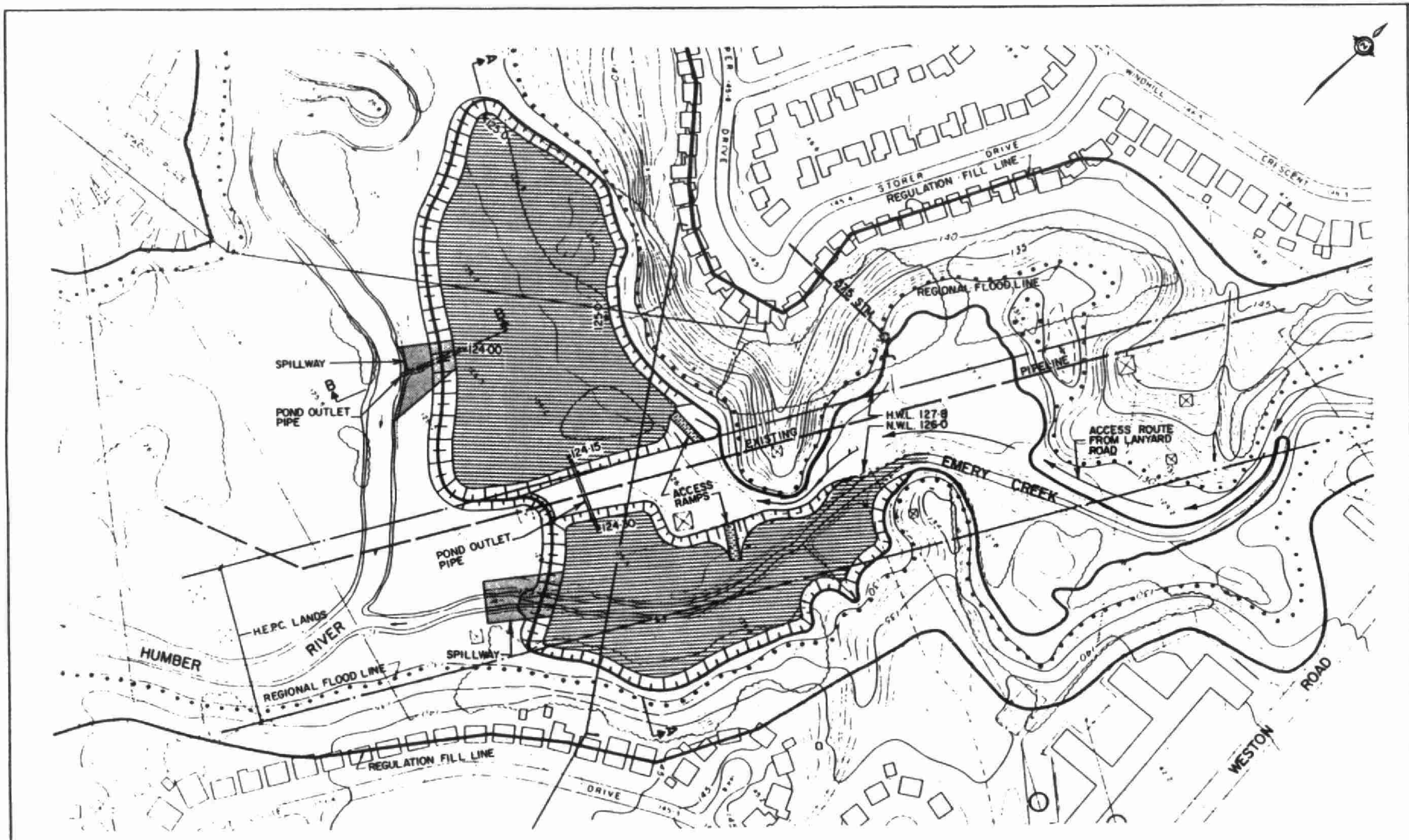
Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

STORMWATER RETENTION PONDS CATCHMENT AREAS

legend



Feasibillity Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

STORMWATER RETENTION POND SITE PLAN

St. Lucie Drive And Storer Drive, North York

0 25 50 100 150
scale metres

legend

⊠	existing hydro tower
147·5	proposed elevation
H.W.L.	high water level
N.W.L.	normal water level
	limit of normal water level

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Feasibility Study And Costing Of Proposed Pollution Control Measures In The Humber Sewershed

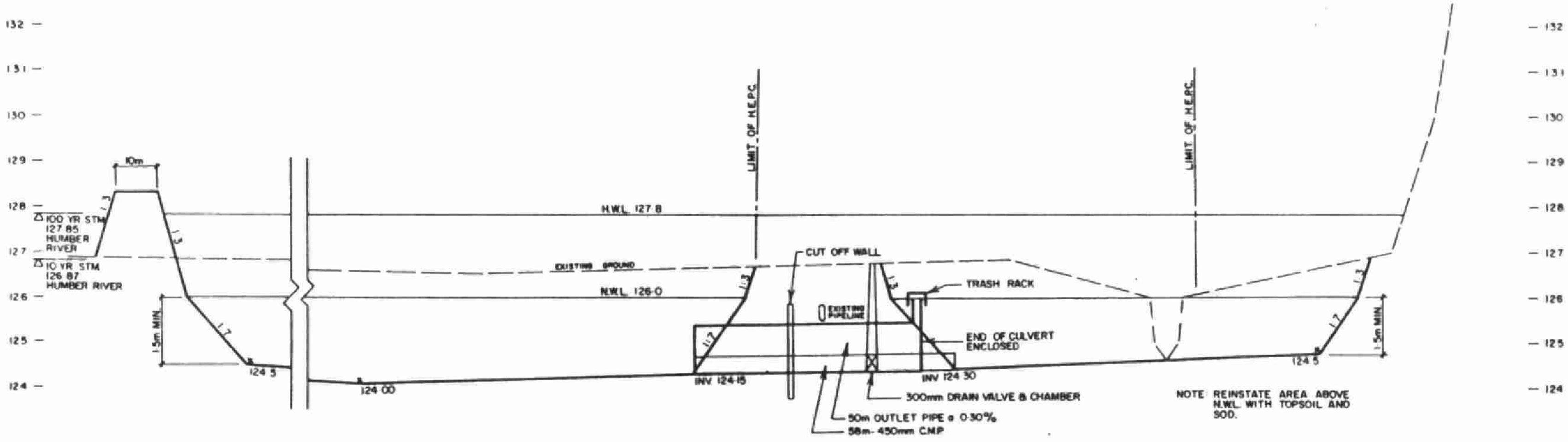
STORMWATER RETENTION POND SECTIONS AND DETAILS

St. Lucie Drive And Storer Drive, North York

scale: 0 1 2 3 4 5 vertical; 0 10 20 30 40 50 horizontal; metres

legend

H.W.L	high water level
N.W.L	normal water level



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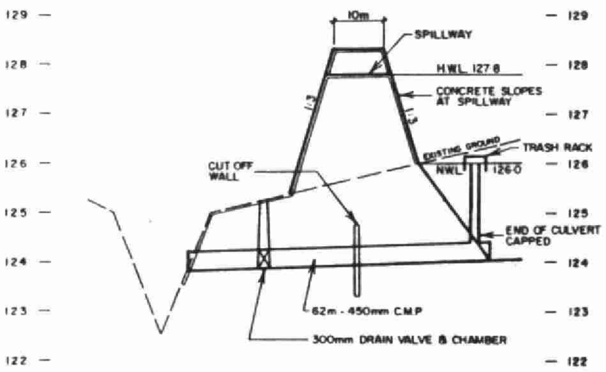

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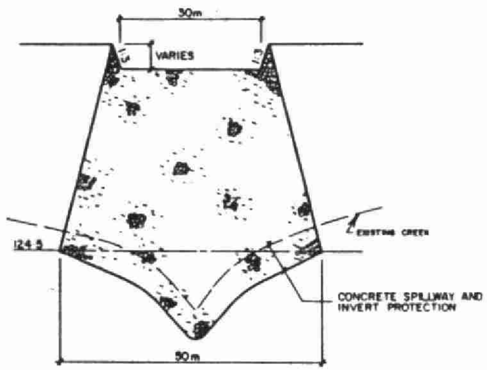
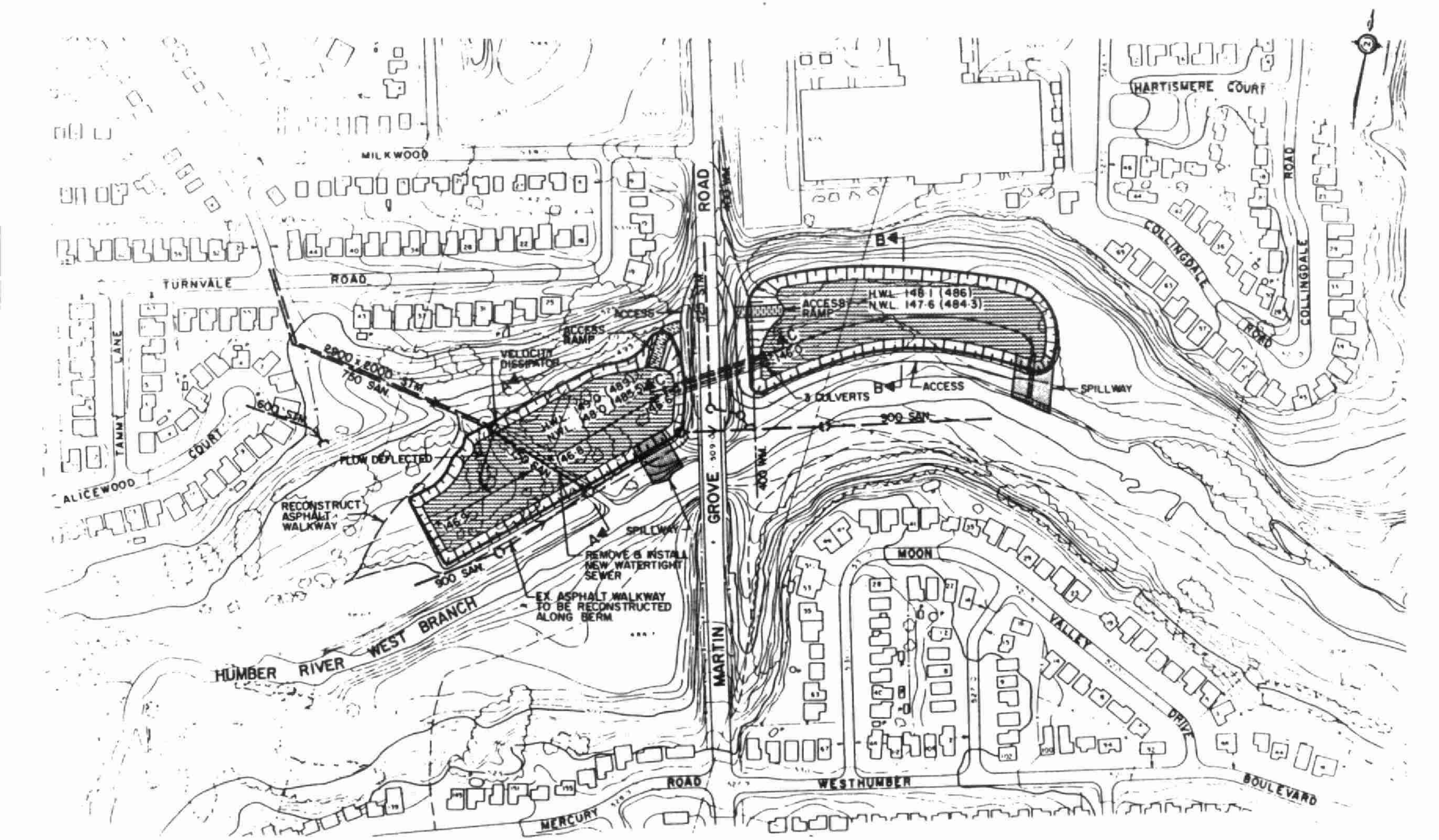


SPILLWAY



Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

STORMWATER RETENTION POND SITE PLAN

Martin Grove Road And Westhumber Blvd., Etobicoke

0 25 50 100 150

scale metres

legend

900	existing sewer (mm)
147.6	proposed elevation (m)
(484.3)	proposed elevation (feet)
H.W.L.	high water level
N.W.L.	normal water level
	limit of normal water level

Note: contours are in feet.

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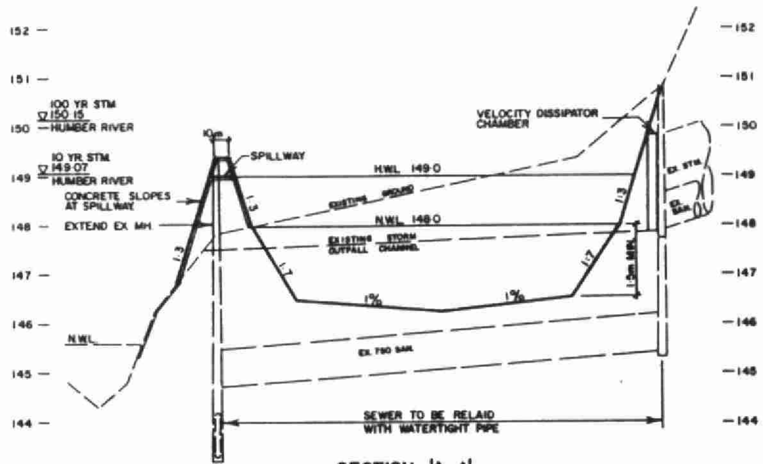

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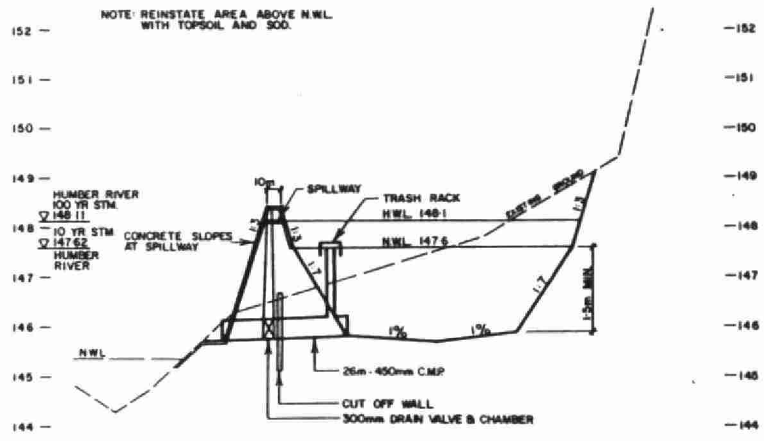

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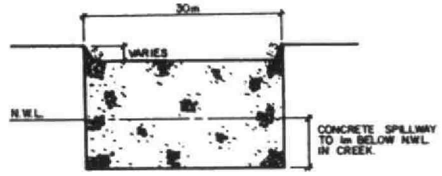
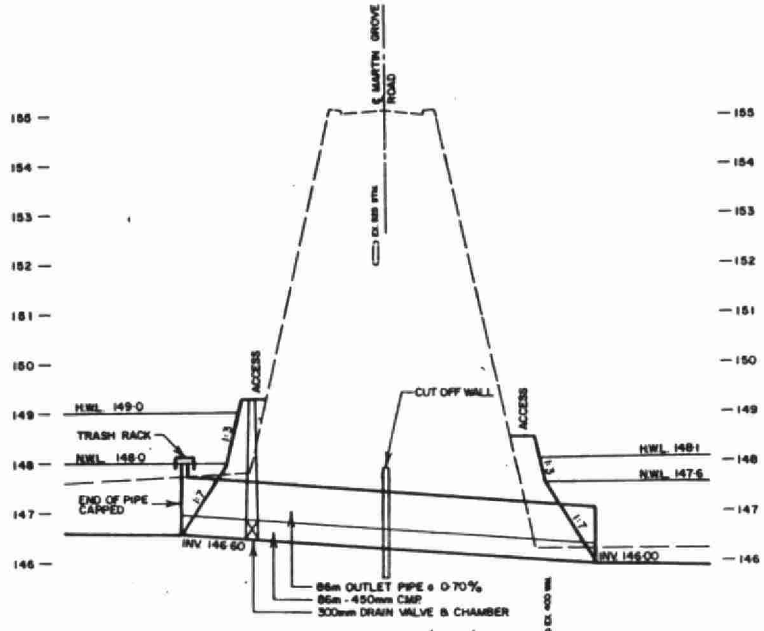


SPILLWAY



SECTION 'C-C'

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Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

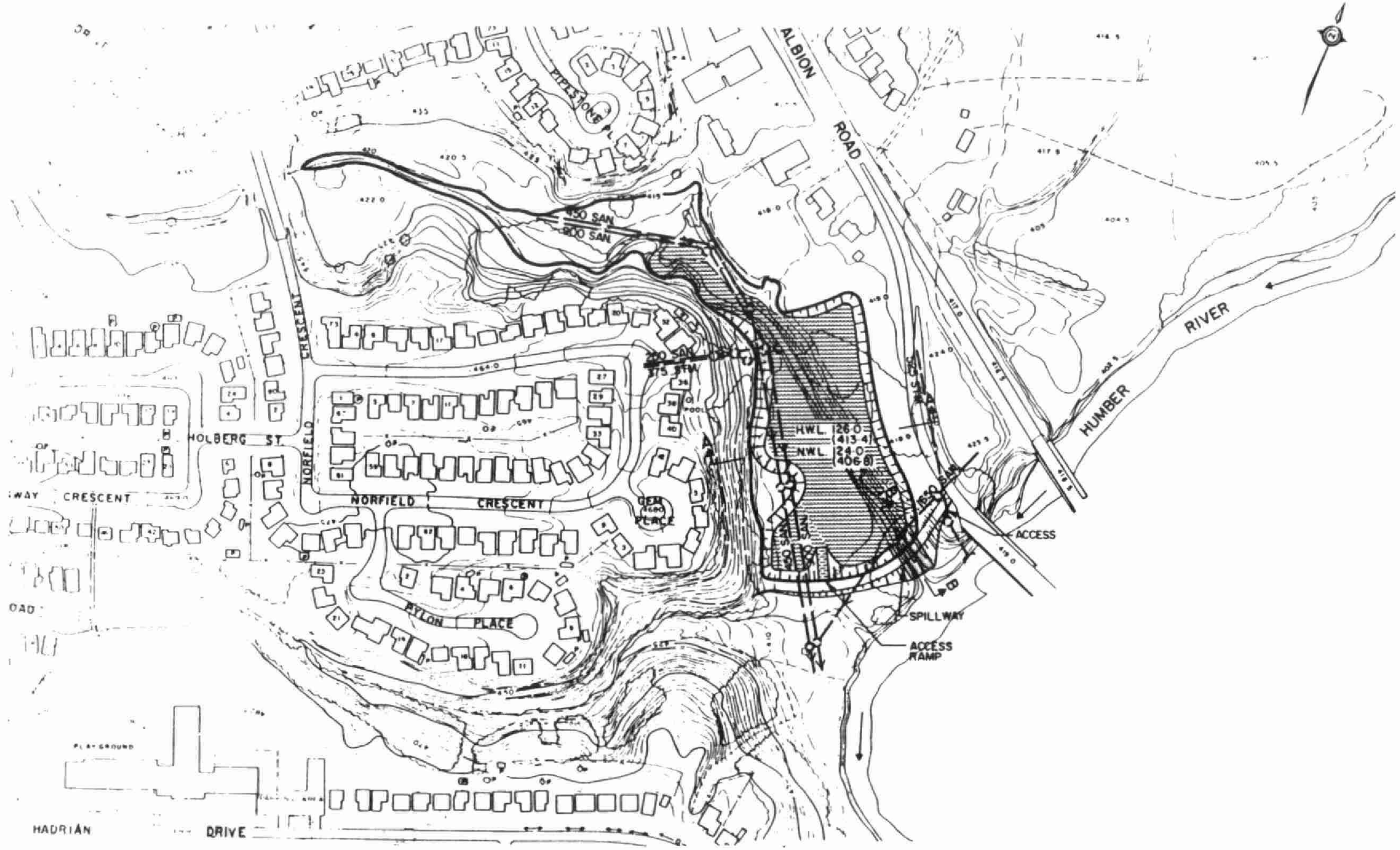
STORMWATER RETENTION POND SECTIONS AND DETAILS

Martin Grove Road And Westhumber Blvd., Etobicoke

0 1 2 3 4 5 vertical
0 10 20 30 40 50 horizontal
scale metres

legend

H.W.L.	high water level
N.W.L.	normal water level



paul theil associates limited

Feasibility Study And Costing Of

Proposed Pollution Control Measures In The Humber Sewershed

STORMWATER RETENTION POND SITE PLAN

Albion Road And Norfield Crescent, Etobicoke

scale 0 25 50 100 150 metres

legend

900	existing sewer (mm)
124.0	proposed elevation (m)
(406.8)	proposed elevation (feet)
HWL	high water level
NWL	normal water level
	limit of normal water level

Note: contours are in feet